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# Tests of bolted beam-to-column flange moment connections, May 1975

K.F. Standig

G.P. Rentschler

W.F. Chen

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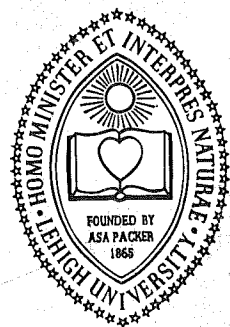
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**BEAM-TO-COLUMN CONNECTIONS**

**TESTS OF  
BOLTED BEAM-TO-COLUMN  
FLANGE MOMENT CONNECTIONS**

**BY  
KENNETH F. STANDIG  
GLENN P. RENTSCHLER  
WAI-FAH CHEN**

**FRITZ ENGINEERING LABORATORY REPORT No. 333.31A**

LEHIGH UNIVERSITY  
BETHLEHEM, PENNSYLVANIA 18015

DEPARTMENT OF CIVIL ENGINEERING  
FRITZ ENGINEERING LABORATORY #13

333/405

November 26, 1975

M E M O R A N D U M

TO: Members, WRC Task Group on Beam-to-Column Connections  
J. A. Gilligan (Chrm.) C. F. Diefenderfer C. L. Kreidler  
V. V. Bertero N. W. Edwards H. A. Krentz  
O. W. Blodgett W. E. Edwards W. A. Milek, Jr.  
V. R. Cartelli H. J. Engstrom, Jr. C. W. Pinkham  
H. C. Crick T. R. Higgins A. N. Sherbourne  
I. M. Hooper F. W. Stockwell, Jr.

FROM: W. F. Chen

SUBJECT: F.L. Report 333.31A--Tests of Bolted Beam-to-Column Flange Moment Connections

Enclosed is Fritz Laboratory Report 333.31A. This is a revised version of Report 333.31 which was discussed at the Welding Research Council Task Group meeting at Lehigh University in July.

At that meeting, a request was made of the members to submit any modifications or additions to Report 333.31 to the Lehigh staff. These modifications and additions would be incorporated into the report for submission to the Welding Research Council for publication. The enclosed report No. 333.31A includes such comments made by Task Group members plus some material contributed by the Lehigh staff.

Please review the enclosed report and send any comments you may have to me. In reviewing the report, I direct your attention to pages 21 through 27 which contain the bulk of the additions and modifications.

If we do not receive your comments within two weeks, we assume everything is acceptable to you. We will then forward the report to WRC for publication.

  
W. F. Chen

WFC:mmm

Enclosure

cc: E. W. Gradt H. Bandel  
I. M. Viest A. C. Kuentz  
C. R. Felmley, Jr. R. Bjørhovde  
W. C. Hansell ✓ Project Staff



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April 30, 1971

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A Guide and Commentary

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Gentlemen:

Under separate cover I am sending each of you a copy of the Second Edition of ASCE Manual No. 41, PLASTIC DESIGN IN STEEL - A GUIDE AND COMMENTARY. I am certain that this publication, accomplished as a result of the work of your committee, will be of significant value to the profession.

Sincerely,

William H. Wisely  
Executive Director

Beam-to-Column Connections

TESTS OF BOLTED BEAM-TO-COLUMN FLANGE MOMENT CONNECTIONS

by

Kenneth F. Standig

Glenn P. Rentschler

Wai-Fah Chen

This work has been carried out as part of an investigation sponsored jointly by the American Iron and Steel Institute, American Institute of Steel Construction and the Welding Research Council.

Department of Civil Engineering

Fritz Engineering Laboratory  
Lehigh University  
Bethlehem, Pennsylvania

May 1975

Fritz Engineering Laboratory Report No. 333.31A

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ABSTRACT

A test program consisting of 12 full size specimens was recently completed which had as its objective the investigation of various welded and/or bolted symmetrically-loaded moment-resisting beam-to-column connections. These connections are of importance in the design and construction of steel multi-story frames. All specimens were fabricated of ASTM A572, Grade 55 steel. This report discusses the results of tests on four specimens of moment resisting bolted connections. All bolts were high strength bolts. In three specimens bolts were designed to be in bearing and in one specimen bolts were designed to make the connection slip resistant (friction). Test results show that the bearing type bolted connections initially behave similarly to fully-welded connections of previous tests. However, at a certain "slip" load, the bolted connections exhibit a distinct difference from the welded connections in overall behavior. The "slip" load at which this occurs can be predicted. The maximum load-carrying capacity of bolted connections is found considerably higher than that of welded connections. Against the background of these results, new design procedures for shear plate, moment plate, column web stiffeners and their welding requirements to column web are suggested. These new procedures provide good correlation with the test results.

## 1. INTRODUCTION

One of the most important components of multi-story building frames is the moment-resisting beam-to-column connection. Most of the past research on beam-to-column connections was performed on welded or riveted specimens. Welded connections are often used in plastically designed structures. The vertical groove welds on this type of connection can be expensive in the field. In recent years, ASTM A325 and A490 high strength bolts have become popular for field erection. The advantages of bolted connections and combinations of welding and bolts became more apparent because of their reduced fabrication and erection costs, when compared with the welded connections. As part of a continuing research project at Lehigh University, the behavior of such connections is being investigated, to provide information for design provisions for such connections.

Full-scale tests were conducted on twelve specimens representing interior beam-to-column moment connections. The objective of these tests was to examine the overall behavior of the connection, and the stresses developed at specific locations. The types of connections in the total project include fully-welded, flange welded with various means of carrying the shear load, and fully bolted.<sup>1</sup> The results of tests on fully-welded and flange-welded, web bolted connections were reported in WRC Bulletin 188.<sup>2,3</sup> Test results for connections which are flange welded, with the means of carrying the shear load varied, were recently reported.<sup>4</sup>

All the research presently underway at Lehigh University on beam-to-column connections is being done for the condition of static

loading. Cyclic loading of similar connections was investigated by Popov and Pinkney.<sup>5</sup>

The present test series, designated as C-series (Table 1), included specimens representing connections for the lower stories (W27x94 beams connected to W14x176 column), middle stories (W24x61 beams, W14x136 column), and upper stories (W14x74 beams, W10x60 column) of a multi-story frame. Each group includes a fully groove-welded connection to act as a control specimen for comparisons.

This paper presents the results of tests on the four fully-bolted specimens (C6, C7, C8 and C9), and compares them with results of other types of connections (see Table 1 and Refs. 2, 3 and 4). The specimen types tested are 1) flange bearing bolted, web bearing bolted (C7), 2) flange bearing bolted, with a stiffened beam seat (C6) (both of these types using W14x74 beams and W10x60 columns), 3) flange bearing bolted, web bearing bolted (C9), and 4) flange friction bolted, web bearing bolted (C8) (both of these types using W24x61 beams and W14x136 columns).

## 2. SPECIMEN DESIGN AND TEST

### 2.1 Design Concepts

The connections were designed according to the AISC Specifications<sup>6</sup> (Appendix 1 of this paper gives typical design calculations), except that the allowable shear stress on A490 bearing bolts was taken as 40 ksi (276 MN/m<sup>2</sup>), and the allowable shear stress on A325 bearing bolts was taken as 30 ksi (207 MN/m<sup>2</sup>), according to the recent recommendation of Refs. 8 and 9. All welding followed the AWS Specification.<sup>7</sup>

The test specimens were designed so that the plastic moment of the beam section,  $M_p$ , and the factored shear capacity of the connection would be reached simultaneously (Fig. 1). The factored shear load is obtained by determining the number of high strength bolts which can be placed in a single line in the beam web, and multiplying that capacity by the factor 1.7. The beam length is then determined from the ratio of its plastic moment to the factored shear load.

In designing the connection, the total moment in the beam at the face of the column is assumed to be transferred to the column as a couple through the beam flanges. Moment plates groove-welded to the column flange pick up the couple as axial forces, through high strength bolts. In the smaller sized connections, the calculated force transferred to the column through the moment plates, both tension and compression, exceeded the capacity of the column web, according to the AISC Specification (Eq. 1.15-1). Therefore, horizontal stiffeners were provided in the column web, according to Eq. 1.15-4 of the AISC Specifications. These stiffeners were fillet-welded only to the column flanges. The

column shapes of both the intermediate and larger size connections were chosen to be the lightest shape which could be used without stiffeners.

ASTM A572, Grade 55 steel was used in fabrication of all specimens. High strength steel was used because the ratio of yield and tensile strengths is lower than that of lower-strength steels. The plastic range of A572 steel is somewhat less than that of A36 steel. Thus, if the connection behavior is adequate for A572 steel, the results could be assumed to apply to lower strength steels.

## 2.2 Bolted Specimens

This paper is specifically concerned with the flange bolted connections under Phase 11 of the test series (Table 1), labeled C6, C7, C8 and C9.

Figure 2 shows Test Specimen C6, a flange-bolted connection with a stiffened beam seat. The design procedure follows the example given on page 4-92 of the AISC Manual,<sup>6</sup> and is in Appendix 1 of this paper. However, strict adherence to the method in the AISC example would require ten bolts in each beam flange, instead of the eight provided (even considering the higher allowable stresses used for the high strength bolts). The lower number was used because of the necessity of having an even number of bolts on each flange without severely overdesigning the bolted flange joints. This should be kept in mind in analyzing the results.

Specimen C6 connects W14x74 beams to a W10x60 column. It is designed to resist moment through a bearing-type connection using 1 in.

diameter A490 bolts in 1-1/16 in. holes. The moment plates are groove-welded to the column flange. Shear is carried by a stiffened beam seat fillet-welded to the column flange. Horizontal stiffeners are provided between the column flanges, and are fillet-welded to the column flanges, but not to the column web.

Test C7 in Fig. 3 is similar to C6. The only difference is that in C7 the shear is carried by 1 in. diameter A490 bolts in the beam web. They are designed for bearing. The shear plate on only one side of the beam is fillet-welded to the column flange, and the eccentricity of the shear connection is neglected in the design of the fillet weld.

Tests C6 and C7 are from the smallest beam-column combination studied. The results of Tests C10 (fully-welded) and C1 (flange-welded, web bolted) have already been reported,<sup>2</sup> and because those specimens are of the same size, the results should be comparable to those reported here.

Test Specimen C8 in Fig. 4 is of the connection of W24x61 beams to a W14x136 column. The moment is resisted by a friction type connection having 1½ in. oversize holes in the moment plate. The use of 1½ in. round holes for 1 in. diameter A490 bolts is permitted by the Specification,<sup>10</sup> because there is no reduction of slip resistance of the joint. The moment plates are groove-welded to the column flange. A bearing connection using 3/4 in. diameter A325 bolts in long slotted holes is used to resist shear. The shear plate, on only one side of the beam, is fillet-welded to the column flange. As before, the eccentricity of the shear connection is ignored in the design of the vertical fillet weld.

Figure 5 shows Test Specimen C9. It is similar to C8, but for the purpose of comparison was designed as a bearing type connection for resisting moment, meaning that fewer bolts were required in each beam flange. Again, as with the design of tests C6 and C7, fewer bolts were provided in each beam flange than required to carry the full plastic moment of the beam.

The control test for C8 and C9 is the fully-welded test specimen C11, reported on in WRC Bulletin 188.<sup>2</sup>

### 2.3 Mechanical Properties

The material used for both beams and columns is ASTM A572 Grade 55 steel. The properties used in determining stresses are the mean values obtained from tension tests performed on specimens of the actual materials. They are as follows:

Modulus of elasticity ( $E$ ) = 29600 ksi (204000 MN/m<sup>2</sup>)

Yield strain ( $\epsilon_y$ ) = 0.00186 in./in.

Yield stress ( $\sigma_y$ ) = 54.9 ksi (379 MN/m<sup>2</sup>)

Strain at the onset of strain hardening ( $\epsilon_{st}$ ) = 0.0150 in./in.

Strain hardening modulus ( $E_{st}$ ) = 581 ksi (4000 MN/m<sup>2</sup>)

A detailed report of mechanical properties is included in Ref. 11.

### 2.4 Instrumentation

The overall instrumentation on all four bolted specimens was similar.



SR-4 linear strain gages were attached to the flanges at the upper portion of the column for use in aligning the specimen under the testing machine crosshead. SR-4 strain gages were also placed on beam flanges to provide checks for possible lateral buckling, and to measure the strain distribution.

Deflection dial gages were located directly under the column for measuring overall deflection (Fig. 1), and in the panel zone to measure any panel zone deformation. In the larger two specimens (C8 and C9), a dial gage was placed in the column web compression region for measuring lateral displacement of the column web.

Rotation gages were used at the junction of ~~the beam~~ and the column to measure the rotation of the connection and also at the beam supports.

Figure 6 shows the general location of the strain gages. Two lines of strain gages in the beam web obtain the strain distribution in the beam at the free end of the moment plate (line A), and at the column face (line B). Strain gages on the moment plates measure the strain distribution at certain lines of bolts. The strain gages in the column web panel zone were placed to provide the general strain distribution throughout the zone. Strain gages were placed at  $\frac{1}{2}(t_p + 5k)$  from the centerline of the moment plates to provide data which, along with later tests, should determine the validity of present assumptions of stress distribution at the k-line in the column web.<sup>12</sup> For this reason, all strain gages shown along the column inner face were placed at the toe of the fillet or the k-line.

Strain gages were also placed on the horizontal stiffeners of C6 and C7, the beam seat stiffener of C6, and the web plates of C8 and C9.

## 2.5 Test Setup

A 5,000,000 lb (22,250,000 N) capacity hydraulic universal testing machine was used to apply axial load in the column as shown in Figs. 1 and 7. The beams were supported by two pedestals resting on the floor. Rollers were used to simulate simply-supported end conditions. Because of the size of sections and the short span of the beam used, no lateral bracing was needed to provide stability. Bearing stiffeners were provided over supports to prevent crippling of the beam web.

## 2.6 Test Procedure

The applied load was increased in increments until failure. The only sources of unloading were due to the specimen itself (slip or bolt failure). Load increments for all specimens were initially 50 kips (223 kN), then decreased to 25 kips (111 kN), and then to increments controlled by a column deflection of about 1/10 in. (2.54 mm). After each load increment, all gage readings were recorded. Points of a load-deflection (P- $\Delta$ ) curve were plotted continuously so that general specimen behavior could be observed and load increments adjusted.

Individual occurrences of slip of the moment plates into bearing on the bolts was accompanied by greater deflection, and in some cases, substantial unloading of the specimen. On the load deflection curves referred to later, the effects of slip are averaged by plotting

only increasing load and not plotting any of the horizontal slip plateaus. The magnitude of these slips was small, and there was no single major slip.

### 3. TEST RESULTS AND DISCUSSION

#### 3.1 Bolted Tests Highlights

##### C6 - Flange Bearing Bolted with Beam Seat

Photographs after testing of specimen C6 are shown in Fig. 8 through Fig. 12. Figure 8 shows an overall view of C6 at the conclusion of testing depicting the extensive yielding in the beam web extending to the end of the moment plates. Failure of this test occurred at a load of 478.5 kips (2129 kN) when the eight bolts connecting the tension flange to the moment plate sheared off simultaneously. Figure 9 presents a closer look at the beam on which the eight bolts fractured. Figure 10 shows both the yielding that occurred in that flange and the deformation at the bolt holes.

Shown in Fig. 11 is a view of a portion of the column panel zone adjacent to the beam tension flanges. Figure 12 shows the shear failure of four of the bolts and their extensive deformation at failure.

##### C7 - Flange Bearing Bolted, Web Bearing Bolted

Photographs of specimen C7 after testing are shown in Fig. 13 through Fig. 17. Figures 13 and 14 show the extensive yielding in the beam web extending to the end of the moment plates. Failure of this test was due to the fracture of five bolts in one of the tension flange connections. Two outboard bolts fractured in this flange; one at a load of 295 kips (1313 kN) the other at a load of 387 kips (1722 kN), due to the prying action of the moment plates which can be seen in Fig. 15. Three more bolts fractured in the same flange at the maximum load of 450 kips (2003 kN).

Figure 16 is a view showing yielding of the column panel zone and the rotation of the beam web. Figure 17 shows the shear plate with the bolts removed. These bolts were easily removed, indicating that they carried little of the applied moment. Figure 18 shows the five bolts which fractured.

#### C8 - Flange Friction Bolted, Web Bearing Bolted

Photographs showing specimen C8 after testing are shown in Fig. 19 through Fig. 22. At the maximum load of 516 kips (2296 kN), the load dropped off due to column web buckling and the test was terminated for safety reasons. Figure 19 shows an overall view of C8 at the termination of testing.

Shown in Fig. 20 is a view of one of the beams at its connection to the column. The appearance of slip on the tension flange (top) can be detected by the displacement of originally vertical lines. Slip in the compression flange (bottom) was much less. Figure 21 shows the extensive yielding which occurred in the column panel zone and Fig. 22 shows the extent of the buckling of the column web in the compression region (a straight piece of metal has been hung against the column web to show this).

#### C9 - Flange Bearing Bolted, Web Bearing Bolted

Photographs showing specimen C9 after testing are given in Fig. 23 through Fig. 25. The maximum load on this connection was 402 kips (1789 kN). Failure occurred when the six bolts on one of the tension flange connections and one of the web bolts sheared off simultaneously. This can be seen clearly in Fig. 23.

Shown in Fig. 24 is an overall view of the specimen after testing and Fig. 25 is a view of the column panel zone and beam ends showing the yield pattern.

### 3.2 Load-Deflection Behavior

#### 3.2.1 Overall Behavior

The overall behavior of the beam-to-column connections studied, which has been characterized by load-deflection curves, is of major interest to engineers and will enable the development of improved design rules. The curves reflect joint 1) strength, 2) rotation and 3) stiffness. These are the criteria which must be examined to insure an adequate connection.

With the completion of the tests on the bolted connections, the effect of each change in connection design can be seen.

The first specimen tested and fully analyzed was C12, a fully-welded connection of the largest size considered in this study.<sup>3</sup> By comparing the behavior of C10, C11 (also fully-welded, but of different size) and C12 on a non-dimensionalized plot, Fig. 26, it can be seen that their overall behavior is similar. The fully-welded specimens may be considered the "ideal connections" because they involve the welded connection of metal to metal. Consequently, they are used as the control specimens for their respective size groups.

The first departure from the fully-welded connection was elimination of the vertical weld on the beam web. This weld can be expensive when done as a field weld. Specimen C1 employed a bolted web

plate to carry shear, in place of the vertical weld, and behaved very much like C10, its control, as shown in Fig. 27. The difference between C1 and C10 is small. Specimen C4, which was designed with a stiffened beam seat to carry shear, behaved much the same as C11, its control, when retested<sup>4</sup> (Fig. 28). The retest was necessary because in the original test the beam web buckled in the region over the beam seat. Beam web stiffeners were added to the specimen for the retest. Specimen C5, also in Fig. 28, relied solely on the groove welds on the beam flanges to carry both moment and shear. It exhibited much lower strength than the other specimens.<sup>4</sup>

The results summarized thus far have been for tests on connection specimens which carried the moment by the welding of the beam flanges to the column flange. The tests recently completed, and reported now, carried the applied moment by the bolting of the beam flanges to moment plates which were welded to the column flange.

Of the bolted moment connections tested, three were designed as bearing connections (C6, C7, C9) and one as a friction (slip resistant) connection (C8).

The load-deflection ( $P-\Delta$ ) curves for specimens C6 and C7 and specimens C8 and C9 are shown in Fig. 29 and Fig. 30, respectively. The dotted lines show the predicted behavior of the connection.  $P_p$  is the value of  $P$  that is theoretically required to cause the plastic moment ( $M_p$ ) to occur in the beam at the column face. The deflection at the intersection of the predicted elastic stiffness line and  $P_p$  is called  $\Delta_p$ . The slope of the upper dotted line indicates the effect when strain

hardening is taken into account. The typical calculations of theoretical load-deflection curves are given in Appendix 2.

From Fig. 29, it can be seen that the behavior of C10, the control specimen, closely follows the predicted stiffness line, and then deviates from it in a smooth curve due to plastic yielding. Initially, C6 and C7 also follow the same predicted stiffness line. However, at a load of approximately 150 kips (668 kN), both specimens abruptly display a smaller, definitely linear stiffness. This is presumed to be due to the slipping into bearing of the high strength bolts in the moment connections and possibly some yielding. At a much higher load, they display a third and smaller slope, due to extensive plastification and strain hardening.

In Fig. 30, C11, the control in this case, follows the prediction well in a smooth curve, as would be expected from the previous comparison of all the fully welded specimens. C9, the bearing connection, follows the prediction until it suddenly follows a smaller slope, beginning at a load of about 185 kips (823 kN). At a higher load, strain hardening begins. It is clearly seen that the moment connections having bolts in bearing behave similarly.

C8, the friction connection, does not display the distinctive second slope due to the slipping of the bolts into bearing (although some slips occurred, identified by the loud bangs accompanying them), but rather follows a smooth curve. The initial stiffness of C8 is slightly greater than predicted due to the longer moment plates not accounted for, which were needed for a friction connection.



### 3.2.2 Theoretical Analysis

The most notable finding in these tests is the distinct difference in overall (P-Δ) behavior between the bolted moment connections designed in bearing and the other connections. The initial load-deflection behavior of all the tests was well predicted. The second linear segment exhibited by the bearing bolted connections is of prime interest. The point of initiation of the second segment of the P-Δ curve, as well as its slope, will now be considered.

The connection of the beam flange to the moment plate will be idealized as a slip-resistant lap joint. Although this is actually a bearing connection, we can consider it initially as a slip resistant joint, because both types exhibit the same stiffness until the load which will cause slip is reached. The bolts in these connections were installed by the turn of nut method, which creates a minimum clamping force that results in initial slip resistance.

The formula for slip load is<sup>9</sup> (Fig. 31):

$$\bar{P}_s = mn T_i k_s$$

where  $\bar{P}_s$  = axial load causing slip

$m$  = number of shear planes

$n$  = number of fasteners

$T_i$  = initial clamping force

$k_s$  = slip coefficient

( $\bar{P}$  is used as the axial load in the plate causing slip, to differentiate it from the load applied by the testing machine.)

Only the three connections designed as bearing types, exhibiting the distinct second slope, are considered. In the calculations done (Table 2) the slip coefficient,  $k_s$ , was taken to be 0.33, an average value. There are no measured values from this series of actual specimens. Initial clamping force,  $T_i$ , was taken to be 64 kips (285 kN) for a 1 in. diameter A490 bolt. This is the minimum fastener tension which will be introduced into the bolt by the use of the turn-of-nut method of installation.

The force which is being carried by the lap joint can be considered to be that force which is in the beam flange at the first row of bolts at the free end of the moment plate (see Appendix 3 for detail calculations). The bending stress in the extreme fiber of the flange is  $M_c/I$  (see Sec. 3.3.2 in this paper), taken at the load applied by the testing machine ( $P$ ) at which the second slope of the  $P$ - $\Delta$  curve initiates. With the simplifying assumption that this stress is constant through the thickness of the flange, the force in the flange ( $Q$ ) is found by multiplying this stress by the area of the flange,  $A_f$ . As can be seen in Table 2, the force in the flange ( $Q$ ) at the load of interest compares well with the slip load ( $\bar{P}$ ) of the idealized lap joint.

Using this same method, the engineer can check for the occurrence of additional deflection due to slip in the connection at working loads. For a completed design of a moment resisting connection utilizing high strength bolts in bearing and moment plates, all that is needed is the calculation of the moment in the beam at the first line of bolts in the moment plates. From this, the force in the beam flange

can be found, which is then compared to the slip load ( $\bar{P}$ ) of the idealized lap joint.

In graphing the second slope on the  $P-\Delta$  curves, the horizontal slip plateaus were averaged, to provide a better view of overall behavior. The specific loads at which individual slips occurred along the second slope are not important and probably could not be reproduced. The importance of the amount of slip is debatable, because slip is in a generally horizontal direction which is constantly changing due to the connection rotation, and the deflection is measured vertically. Shown under the key in Figs. 29 and 30 is a horizontal line illustrating the calculated deflection, in terms of  $\Delta$  which would have occurred had there been one major slip in the moment plate connection rather than the many minor slips which did occur.

The unpredictable individual slips and the specific loads at which these slips occurred, along with yielding at various points in the specimen, makes it difficult to predict the second slope, even though it appears linear. From the data, however, the second slope is seen to be 16% to 23% of the initial elastic stiffness, which can be predicted.

As stated before, the three bearing bolted moment connections were not provided with enough bolts to theoretically allow the plastic moment of the beam to be reached. Table 3 compares theoretical capacities in terms of  $P$ 's, the load applied by the testing machine.

$P_p$  is the load which should cause plastic moment ( $M_p$ ) in the beam at the column face.  $P_b$  is the maximum load which can be applied

based on the capacity of the bolts. Notice that  $P_b$  is lower than  $P_p$ . The allowable stress for A490 bolts was taken as 40 ksi (276 MN/m<sup>2</sup>), as recommended in Refs. 9 and 10, instead of 32 ksi (221 MN/m<sup>2</sup>) specified by AISC.<sup>6</sup> Working load,  $P_w$ , is the maximum load divided by the factor 1.7.  $P_{ult}$  is the maximum load for each specimen, at which failure occurred in each case by the shearing of bolts. The ratio of this maximum load to the working load based on bolt capacity yields the factor of safety (F.S.) of the bolts against failure. These factors of safety are high, even though the design was based on higher allowable stresses for bolts. The factor of safety based on the AISC allowable stresses would be even higher.

At this point it should be noticed that the value of working load based on bolt capacity is comparable to the load at which the second slope initiated on the  $P-\Delta$  curves (for C6 and C7, not C9). If an adequate number of bolts was used originally, the working load would have been controlled by beam capacity, and the second slope would not have initiated until a higher load, a load which is above the region of interest to the practicing engineer.

On the load-rotation relationships, Figs. 32 and 33, all of the bolted connections exhibit good ductility and rotational capacity. The bearing type connections show the same three-segment behavior noticed before.

Figure 34 shows the column web buckling of C8, which caused the failure of that specimen.

### 3.3 Stress Distribution

#### 3.3.1 Column Behavior

Figures 35 and 36 show the horizontal stress ( $\sigma_x$ ) distributions at the column k-line for each test, with the compressive stresses occurring in the upper region and tensile stresses occurring in the lower region. Most of the stresses are transferred over a width of  $t_b + 5k$ , as is specified for use in design. Also, the web plate provided to transfer shear helps in transferring horizontal stresses in a linear distribution, in C7, C8, and C9 contrasted with C6, where there is no connection between the beam web and the column because of the use of a beam seat.

Connection C4 is of the largest size specimen tested, with the beam flanges welded to the column and the shear carried by a stiffened beam seat.<sup>4</sup> Comparing the horizontal stress distribution in C6, with that in C4,<sup>4</sup> it is seen that the compressive stresses in C6 are distributed over a large region, presumably due to the action of the beam seat. In C4, this is not the case. Stresses are concentrated at a level even with the beam flange. The difference is because of the fact that in C6, the moment plate acts as the beam seat and picks up all the compression, which is then distributed through its stiffener. In C4, only an erection bolt directly connects the beam and beam seat not allowing large horizontal stresses to be developed in the stiffener beneath the seat.

Figures 37 and 38 show the vertical stress ( $\sigma_y$ ) distributions at the column innerface. No consistent pattern is in evidence, even

with respect to past test results. However, there is a small biaxial tension zone in C6, C8, and C9.

### 3.3.2 Beam Behavior

Figures 39 and 40 show the horizontal stress variation across a beam section at the end of the moment plates (Fig. 6, Sec. A). This closely approximates the classical  $My/I$  beam stress distribution, and at higher loads begins to look more like a stress distribution representing the formation of a plastic hinge.

Figures 41 and 42 show the horizontal stress variation across a beam section at the column face (or as close to the web plate as possible) (Fig. 6, Sec. B). In C6 and C7, the magnitude of the stresses is less than half those recorded at the end of the moment plates. Also notice that in C6, the entire section is subjected to tensile stresses, suggesting the beam seat stiffener is taking all of the compressive stresses. C9 stresses are of the same order of magnitude as at the end of the moment plates, most probably because of the relatively short plates. C8 stresses are also of half the magnitude discussed before, because of the much longer moment plates required for the friction connection.

This reduction in bending stresses along the length of the moment plates introduces the thought that a plastic hinge, if it forms at all, will form at the end of the moment plates and not at the column face. However, the results from C9 lead to the conclusion that the location of plastic hinge formation will be, in part, dependent upon the ratio of moment plate length to beam depth.

Assuming the plastic hinge in the beam to occur at the end of the moment plate is found unconservative for the theoretical predictions of the connections' maximum or limit loads, while assuming the hinge to be at the column face is too conservative. For the designer who wishes to quickly estimate the maximum load of a bolted connection, assuming the plastic hinge to be at midlength of the flange moment plate will give loads which are conservative but reasonably close to the actual maximum. Using this procedure for the four specimens tested, the predicted loads were all approximately 90% of the actual maximum load.

### 3.4 Other Results

#### 3.4.1 Shear Plates

The yield patterns observed on the beams of C6 and C7 (Figs. 9 and 14) clearly indicate shear yielding. However, the yielding does not extend beyond the end of the moment plates, indicating that the moment plates do resist some shear, although not designed for shear.

Assuming the absence of any beam web shear attachment, calculations were made to check whether the flange moment plates could carry both the shear and bending moment of the connection. Assuming the beam shear to be carried equally by the two moment plates, von Mises yield criterion was used to determine what testing machine load could be applied to the connection before the moment plate steel would yield. Using the net area of the moment plates in all cases, the values calculated ( $P_v$ ) according to the above procedure are shown in Table 4. Also shown are the corresponding design values of  $P_p = 2M_p/L$ , the value of  $P$  theoretically required by simple plastic theory to cause the

plastic hinge or moment ( $M_p$ ) to occur in the beam at the column face. The values of  $P$  assuming the moment plates to carry all shear and moment are higher than the load  $P_p$ , indicating that the flange plates alone can provide enough moment and shear resistance to achieve a load greater than the beam plastic moment. This assumes that the tension-shear resistance of the bolts is sufficient to transfer the load to the plate.

On another specimen in these series of beam-to-column connection tests, the only means of attachment of the beam to the column was by groove-welding the beam flanges to the column flange<sup>4</sup>. The load-deflection plot for this test is shown as C5 in Fig. 28. The absence of shear attachment on this specimen caused the maximum load to be well below  $P_p$ , although there was still sufficient ductility. The reason for the reduced maximum load in this case was that the beam tension flange was torn from the beam web at the flange to web junction, with tearing initiating at the cope in the web. This was due to the concentration of the slightly inclined flange force adjacent to the cope. The vertical component of this inclined flange force causes the tearing. It is believed that this would not happen in bolted flange connections because the vertical component of the inclined flange force and related prying forces are distributed along a length of the beam by the bolts. Thus, the concentration would not be large enough to peel the beam flange from the web.

Therefore, in bolted flange moment connections, it appears theoretically possible to eliminate the normal methods of carrying shear



such as beam web shear plates or beam seats. However, before a firm conclusion can be made in this direction, more research should be conducted to determine how much shear is carried by flange moment plates in bolted flange moment connections.

#### 3.4.2 Stiffeners

The excellent test results of all the specimens verify the adequacy of the present stiffening requirements of AISC Sec. 1.15.5.<sup>6</sup> The results of the unstiffened specimen C8 as shown in Fig. 34 indicate that Formulas 1.15-1 and 1.15-2 are conservative. Formula 1.15-4 appears to be conservative based on the test data of specimens C6 and C7, both connections having column web stiffening. Furthermore, Sec. 1.15.5 implies that column web stiffeners be welded to the column web. The results of specimens C6 and C7 in which the stiffeners are not welded to the column web would indicate that such stiffeners need not be welded to the column web.

#### 3.4.3 Bolts

In the testing of C7, the first two bolts which failed were on the outer line of a tension beam flange. The bolts appeared to fail in tension and a close look at the moment plate shows it bending and probably exerting a prying action on the outer bolts (Fig. 15). This, too, warrants further investigation.

According to Sec. 1.15.6 of the AISC Specifications, shims thicker than  $\frac{1}{4}$  in. are to be developed when used in bearing type joints. In the bearing connections tested (C6, C7, C9) the shims were not extended and developed, with no adverse effects on the strength of the connections.

#### 4. SUMMARY AND CONCLUSIONS

The overall project has studied the behavior of steel beam-to-column moment connections. The behavior of the fully-welded connection considered the ideal case and used as a control is compared with similar fully-bolted connections. The conclusions which can be drawn from the tests of bolted connections are;

1. In contrast to prior tests on moment connections with beam flanges welded to the column, moment connections with fasteners designed for bearing exhibit a slip characteristic that results in a reduction of stiffness at loads less than the plastic limit load of the beam. There are three distinct segments in a typical load deflection curve (Fig. 29).
2. The initial load-deflection behavior agrees well with the theory (Fig. 29). (This prediction neglects stiffening effects of the flange plates.)
3. The load, associated with slip, at which the second segment of the  $P-\Delta$  curve departs from the initial elastic slope can be predicted using lap joint idealization for beam flanges.
4. The bolted moment connections tested exhibit a maximum strength that is at least equal to that of the comparable welded connection and varies up to 30% higher than the welded connection due to the strengthening effect of the moment plates. The higher portion of the  $P-\Delta$  curves are approximately parallel to the strain hardening prediction.
5. Bolted moment connections exhibit sufficient rotational capacity when compared with welded connections.

6. The tests show that the higher allowable stresses of 40 ksi for A490 bolts and 30 ksi for A325 bolts, for connections designed in bearing, provide an adequate factor of safety against their ultimate strength.
7. In bolted connections with moment plates, the plastification of the beam occurs at or beyond the end of the moment plates, and not at the column face. The assumption of the plastic hinge to occur at midlength of the moment plate is found to be adequate for a quick check of connection load carrying capacity.
8. The deflection resulting from slip of bearing bolted moment connections may be an additional factor to be considered in the analysis of the stability of frames.
9. Theoretical calculations indicate that the flange connection plates of bolted flange moment connections can carry both shear and moment. Beam web shear plates or beam seats in bolted connections may be eliminated.
10. The test results have shown the adequacy of the present stiffening requirements of Sec. 1.15.5 of the AISC Manual.<sup>6</sup> When column web stiffening is required, the test data indicates that such stiffeners need not be welded to the column web.
11. Where shims thicker than  $\frac{1}{4}$ " are required between the beam flange and the moment plate, no adverse effects on the strength of the connection were evident when these shims were not developed.

## 5. ACKNOWLEDGMENTS

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6. APPENDICESAPPENDIX 1: DESIGN OF CONNECTION C7

(W14x74 beams and W10x60 column)

## 1. Flange force

$$M_p = \sigma_y Z_x = 6930 \text{ k-in}$$

$$T = \frac{M_p}{d} = 488 \text{ k}$$

## 2. Bolts

Try 1 in. A490-x bolts in 1-1/16 in. round holes. Use allowable stress of 40 ksi. Allowable force/bolt = 31.416 k.

$$n = \frac{T}{(31.416)(1.7)} = 9.14$$

Use 8 bolts, round holes.

pitch = 3 in.

end distance = 1-3/4 in.

plate length = 5 + (3x3) + 1-3/4 - 3/8 = 15-3/8 in.

## 3. Flange moment plate

$$A_p = \frac{T}{F_t(1.7)} = 8.71 \text{ in}^2$$

Try 1-1/8 in. plate

$$b_p = \frac{8.71 + 2(1-1/8 \times 1-1/16)}{1-1/8} = 9.87 \text{ in.}$$

Use 1-1/8 in. x 10 in. x 15-3/8 in. plate. ( $F_y = 55 \text{ ksi}$ )

## 4. Check

Bearing on plate ( $t = 1.125 \text{ in.}$ )

$$\sigma_b = \frac{488}{(1)(1.125)(8)} = 54.2 \text{ ksi}$$

$$\frac{\sigma_b}{\sigma_u} = \frac{54.2}{70} = 0.774$$

$$\frac{0.774}{1.7} = 0.455$$

Bearing on flange ( $t = 0.783$  in.)

$$\sigma_b = \frac{488}{(1)(0.783)(8)} = 78 \text{ ksi}$$

$$\frac{\sigma_b}{\sigma_u} = 1.113$$

$$\frac{1.113}{1.7} = 0.656$$

$$\frac{\lambda}{d} = 1.75$$

#### 5. Stiffeners

$$A_{st} = A_p - t_w(t_b + 5k) = 8.19 \text{ in}^2$$

or  $4.10 \text{ in}^2$  per stiffener.

Use 4 in. x 1 in. plates ( $F_y = 55 \text{ ksi}$ )

Weld size:

minimum  $w = 5/16$  in. (based on  $t = 1$  in.)

$$T = \frac{M}{d} = 488 \text{ k} < \sigma_y(A_p) = 618 \text{ k}$$

$$T_w = F_y t_w(t_b + 5k) = 168 \text{ k}$$

$$2 T_{st} = 488 - 168 = 320 \text{ k}$$

one stiffener:

$$T_{st} = 160 \text{ k} < (T_{st})_y = 220 \text{ k}$$

$$w = \frac{160.0}{(1.7)(2)(21)(.707)(4.0)(1.0)} = .80$$

Use  $7/8$  in. fillet weld, both sides.

APPENDIX 2: THEORETICAL LOAD-DEFLECTION CURVE

Sample calculations for C6 and C7

Beam W14x74	$M_p = 6930 \text{ kip-in.}$
	$I = 797 \text{ in}^4$
	$L = 43 \text{ in.}$
	$d = 14.19 \text{ in.}$
	$t_w = 0.45 \text{ in.}$
Column W10x60	$d_c = 10.25 \text{ in.}$

The load at which a plastic hinge will form in the beam at the column face is

$$P_p = \frac{2M_p}{L} = 322 \text{ kips}$$

For the deflection at which the slope of the elastic behavior will meet the load  $P_p$  ( $\Delta_p$ ), consider bending and shear.

In considering bending, assume the specimen to act as a simple beam of length  $2L + d_c$ . Neglect the change in moment of inertia due to the moment plates ( $E = 29570 \text{ ksi}$ ).

$$\Delta_b = \frac{P\ell}{48 EI} = .2538 \text{ in.}$$

In considering shear, assume a cantilever of length  $L + 1/2 d_c$ , to be compatible with the previous assumption of the length of the simple beam.

$$G = \frac{E}{2(1+\nu)} = 11373 \text{ ksi}$$

$$A_w = (14.19) (.45) = 6.39 \text{ in}^2$$

$$\Delta_v = \frac{V\ell}{A_w G} = .1066 \text{ in.}$$

$$\Delta_p = \Delta_b + \Delta_v = .3604 \text{ in.}$$

APPENDIX 3: CALCULATION OF SLIP LOAD

Sample calculations for C6 and C7. The idealized lap joint is examined at the first line of bolts at the free end of the moment plate. Stress is assumed constant through the thickness of the flange. The force in the flange (Q) is:

$$Q = \frac{Mc}{I} A_f$$

where  $M$  = moment at the first line of bolts in the moment plate  
 $c$  = distance from neutral axis to extreme tension fiber of beam  
 $I$  = moment of inertia of beam  
 $A_f$  = area of beam flange.

From the P- $\Delta$  curve (Fig. 29), the initiation of the second slope due to slip appears to be at a load (P) of 150k. At this load,  $Q = 174$  kips.

The slip load for a lap joint ( $\bar{P}$ ) is given by (see Fig. 31):

$$\bar{P} = mn T_i k_s$$

where  $\bar{P}$  = axial load causing slip  
 $m$  = number of shear planes  
 $n$  = number of fasteners  
 $T_i$  = initial clamping force  
 $k_s$  = slip coefficient

For the configuration of C6 and C7, with  $T_i = 64$  kips and  $k_s = .33$  (an average value),  $\bar{P} = 169$  kips.



There is good correlation between the force required to cause slip in the idealized lap joint, and the force in the beam flange at the load which initiates the second slope of the  $P-\Delta$  curve.

## 7. TABLES

TABLE 1 TEST PROGRAM OF C-SERIES (Ref. 3)

Phase	Test	Beam Size Column Size	Factored Load		Stiffening	Beam Span L
			Moment	Shear		
10	C1	W14x74 W10x60	M = 6930K-in Beam Flange Groove Weld	V = 160K (88.5%V <sub>P</sub> ) Shear Plate 3 <sup>P</sup> -1" φA490-X in 1-1/16" Round Holes	Horizontal Stiffeners 3/4"x4"x 8-7/8"	3'-7"
	C2	W27x94 W14x176	M = 15290K-in Beam Flange Groove Weld	V = 374K (94.7%V <sub>P</sub> ) Shear Plate <sup>P</sup> 7-1" φA490-X in 1-1/16" Rnd.Holes	--	3'-5"
	C3	W27x94 W14x176	DO	DO Shear Plate has Slotted Holes	--	3'-5"
	C4	W27x94 W14x176	DO	V = 374K (94.7%V <sub>P</sub> ) Two-Plate Welded Stiffened Seat	--	3'-5"
	C5	W27x94 W14x176	To Be Estimated Beam Flange Groove Weld	To Be Estimated Beam Flange Groove Weld	--	3'-5"
11	C6	W14x74 W10x60	M = 6930K-in Moment Plates <sup>P</sup> 8-1" φA490-X in 1-1/16" Rnd.Holes	V = 160K (88.5%V <sub>P</sub> ) Stiffener Plate <sup>P</sup> 1"x5"x11"	Horizontal Stiffeners 1"x4"x 8-7/8"	3'-7"
	C7	W14x74 W10x60	DO	V = 160K (88.5%V <sub>P</sub> ) Shear Plate <sup>P</sup> 3-1" φA490-X in 1-1/16" Rnd.Holes	DO	3'-7"
	C8	W24x61 W14x136	M = 8360K-in Moment Plates <sup>P</sup> 14-1" φA490-F in 1-1/4" Rnd.Holes	V = 157.5K (52.5%V <sub>P</sub> ) Shear Plate <sup>P</sup> 7-3/4" φA325-X in Slotted Holes	--	4'-5"
	C9	W24x61 W14x136	M = 8360K-in Moment Plates <sup>P</sup> 6-1" φA490-X in 1-1/16" Rnd.Holes	DO	--	4'-5"
Performance Control Test	C10	W14x74 W10x60	M = 6930K-in Beam Flange Groove Weld	V = 160K (88.5%V <sub>P</sub> ) Beam Web <sup>P</sup> Groove Weld	Horizontal Stiffeners 3/4"x4"x 8-7/8"	3'-7"
	C11	W24x61 W14x136	M = 8360K-in Beam Flange <sup>P</sup> Groove Weld	V = 157.5K (52.5%V <sub>P</sub> ) Beam Web <sup>P</sup> Groove Weld	--	4'-5"
	C12	W27x94 W14x176	M = 15290K-in Beam Flange <sup>P</sup> Groove Weld	V = 374K (94.7%V <sub>P</sub> ) Beam Web <sup>P</sup> Groove Weld	--	3'-5"

TABLE 2 SLIP LOADS (see Appendix 3)

	P	Q	$\bar{P}$	$Q/\bar{P}$
C6 and C7	150	153	169	0.91
C9	185	124	127	0.98

All loads are in kips (1 kip = 4.45 kN)

P = machine load at initiation of slip (from P- $\Delta$  curve)

Q = force in tension beam flange at load P (calculated)

$\bar{P}$  = axial load causing slip in idealized lap joint

TABLE 3 WORKING LOADS

	$P_p$	$P_b$	$P_w$		$P_{ult}$	F.S.
			$\frac{P_p}{1.7}$	$\frac{P_b}{1.7}$		
C6	322	282	189	166	478.5	2.88
C7	322	282	189	166	450	2.71
C9	315	287	185	169	402	2.38

All loads are in kips (1 kip = 4.45 kN)

$$P_p = \frac{2M}{L}$$

$$P_b = \frac{2(1.7)n A_b F_v d}{L}$$

$P_w$  = working load

( $F_v$  = 40.0 ksi or 276 kN/m<sup>2</sup> for A490)

$$F.S. = \frac{P_{ult}}{(P_w)_b}$$

TABLE 4 CONNECTION LOADS ASSUMING FLANGE CONNECTION PLATES TO CARRY ALL  
SHEAR AND MOMENT

	$P_v$ (kips)	$P_p$ (kips)
C6	378.2	322
C7	378.2	322
C8	366.9	315
C9	364.1	315

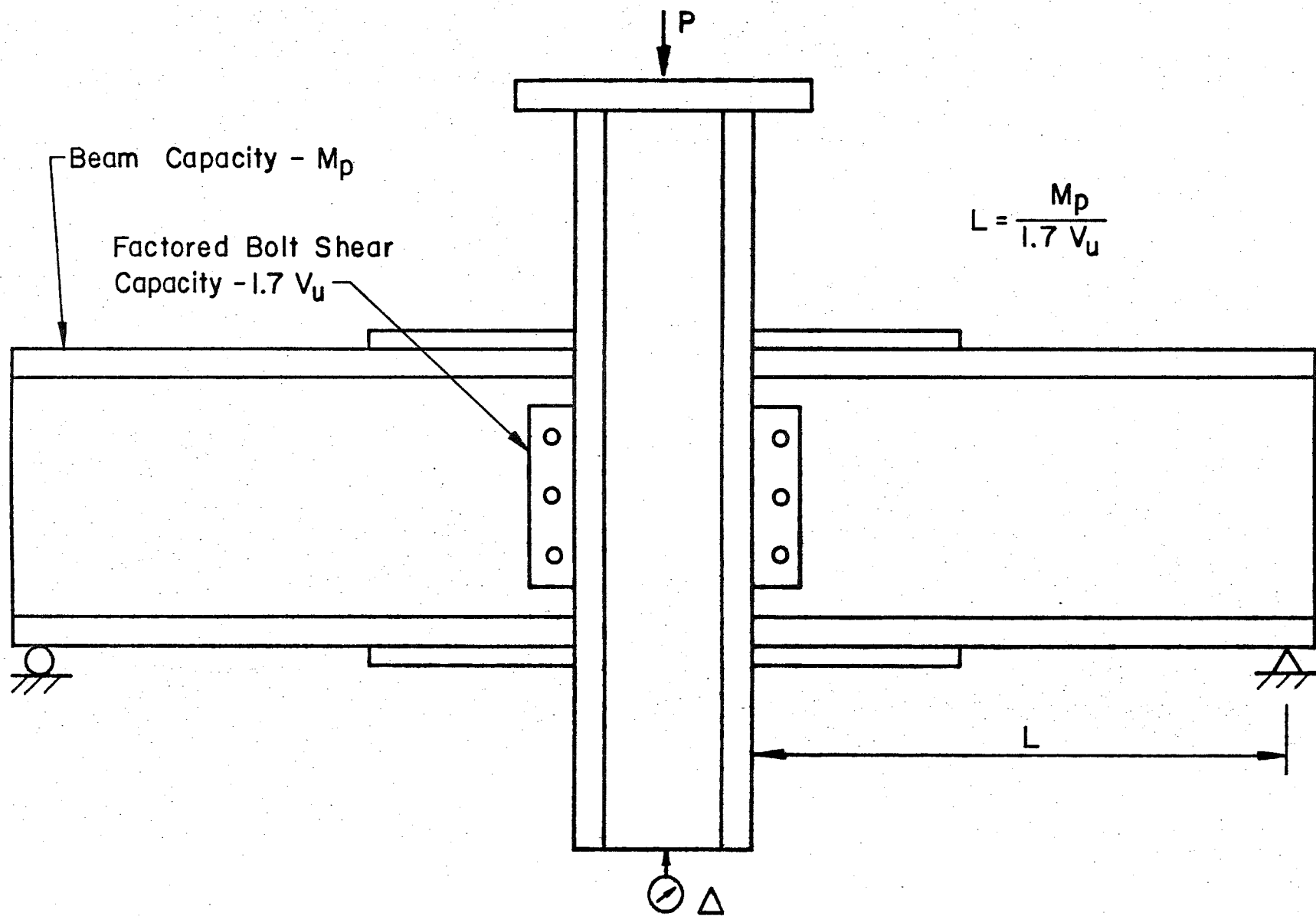
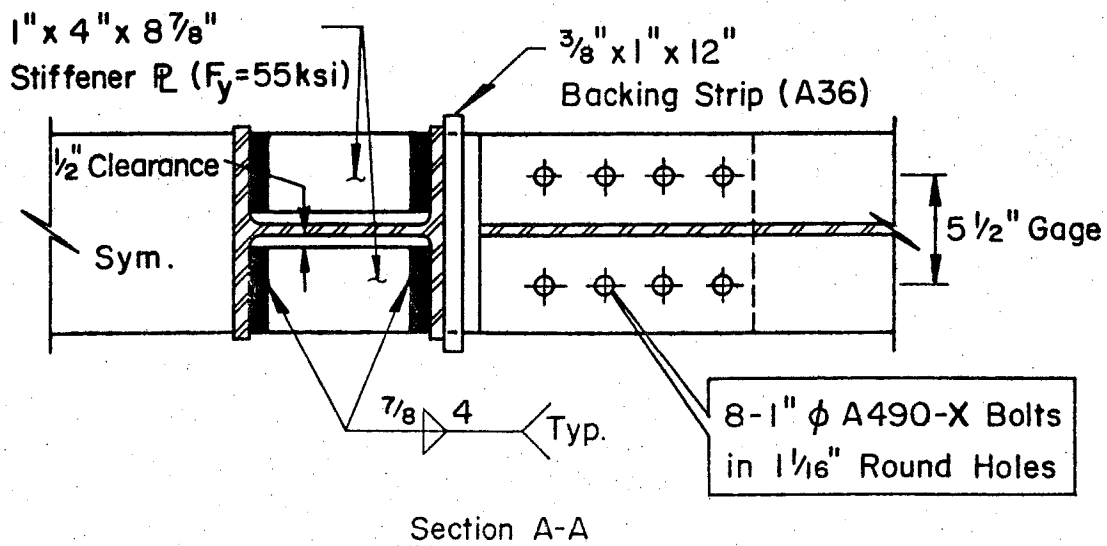
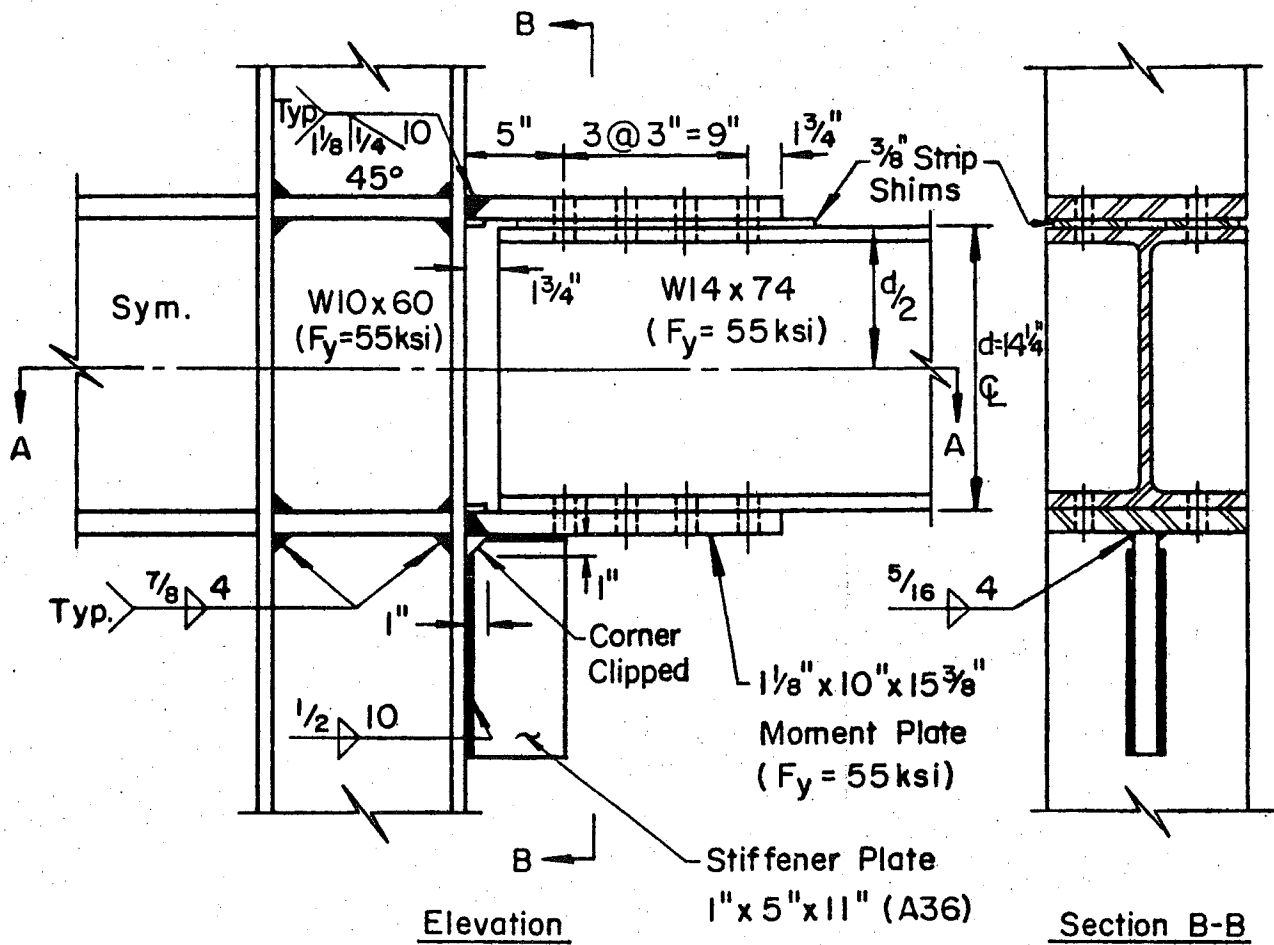


Fig. 1 Specimen Design and Test Setup



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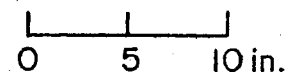
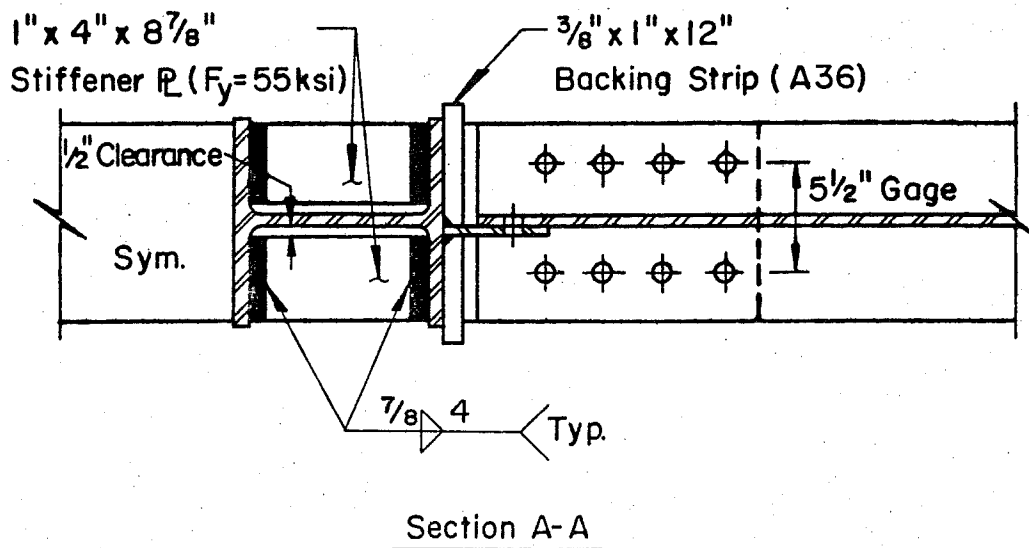
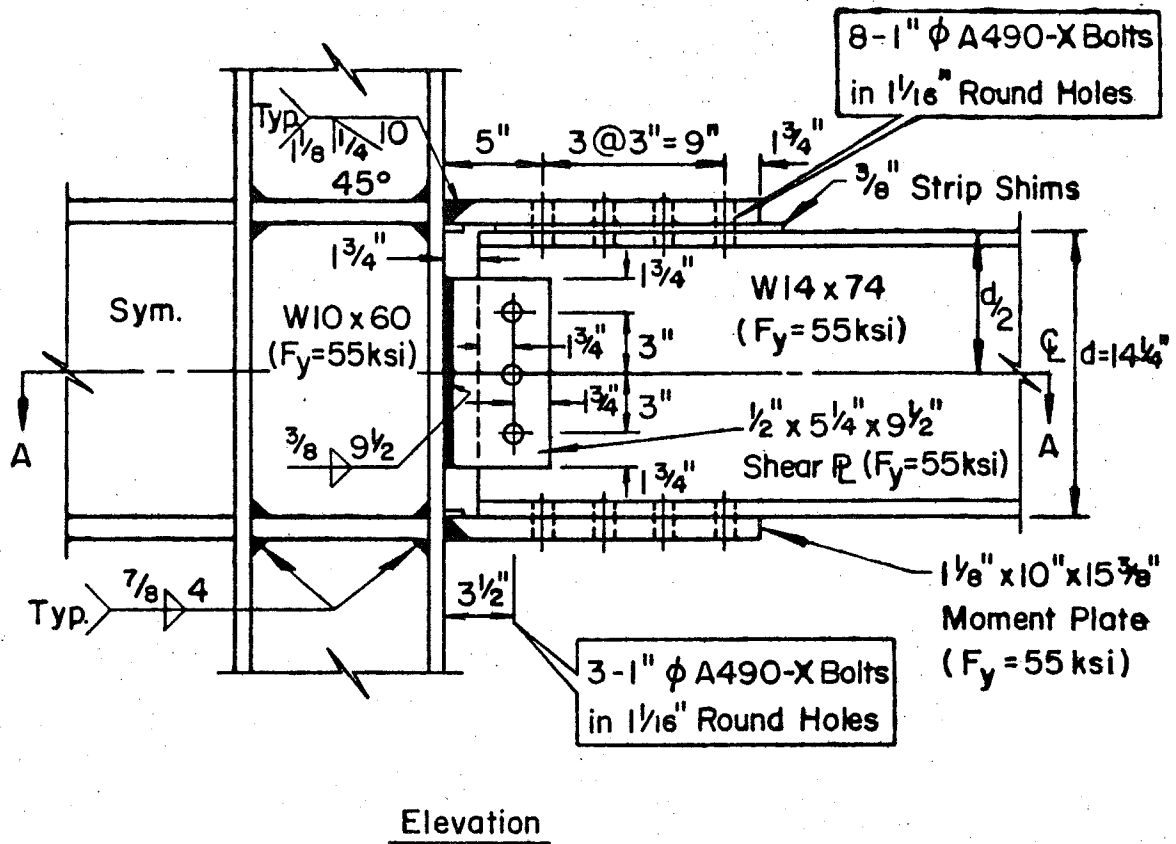


Fig. 2 Test C6



Scale:

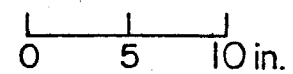
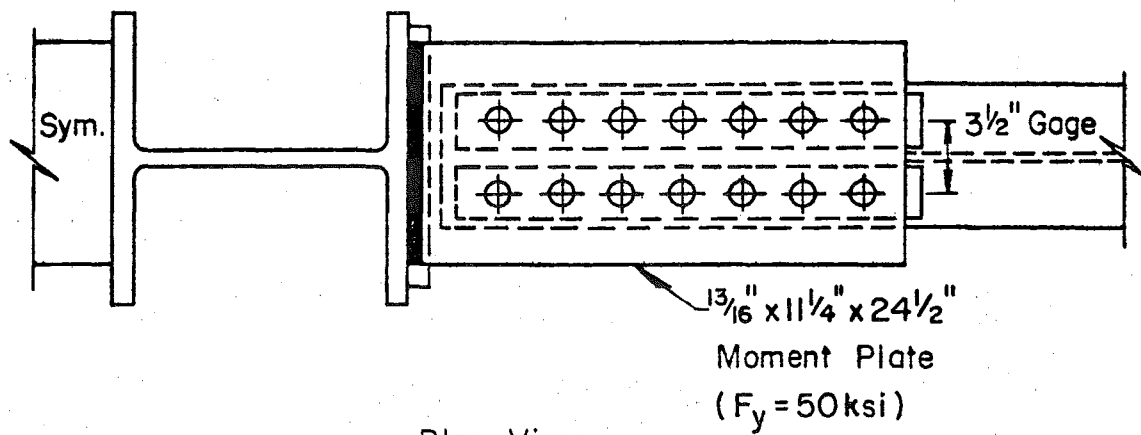
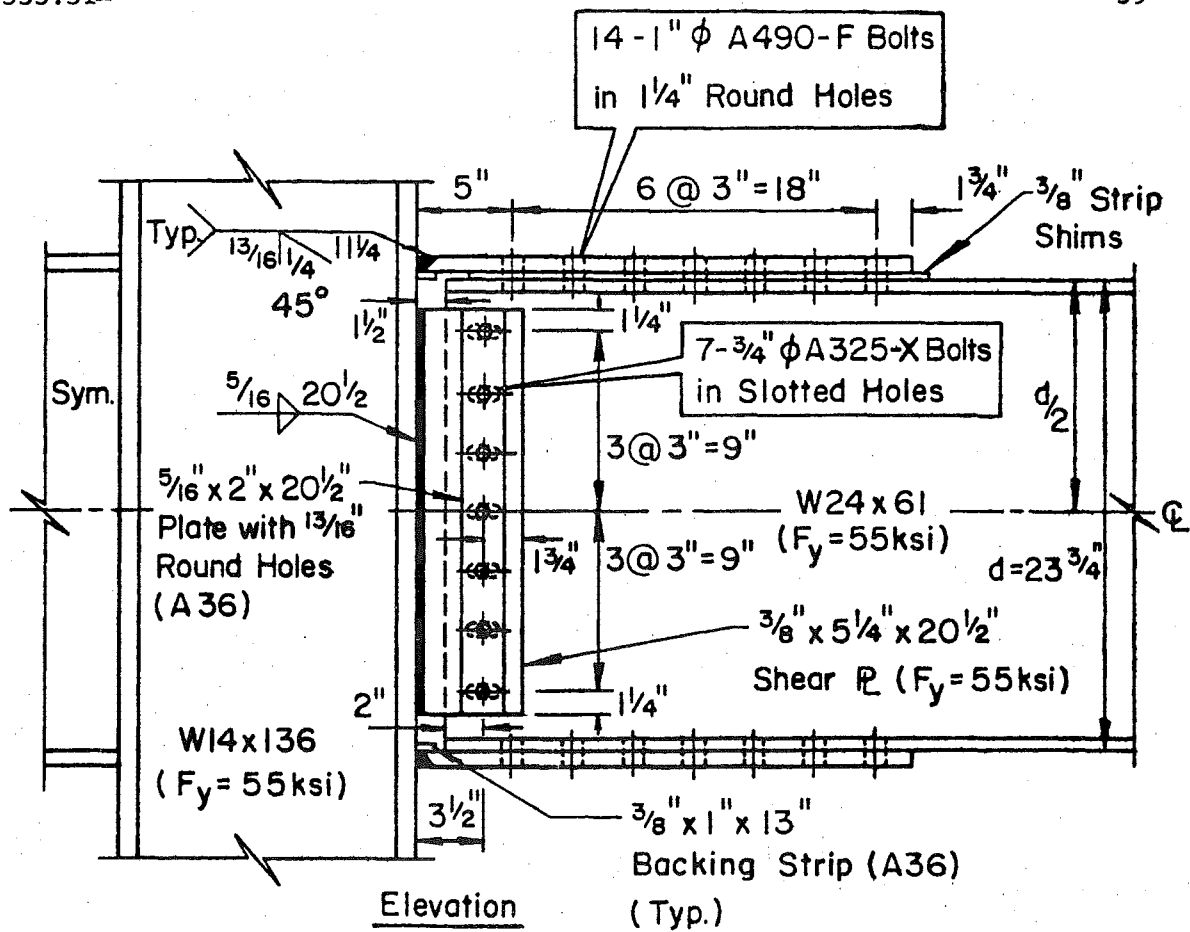
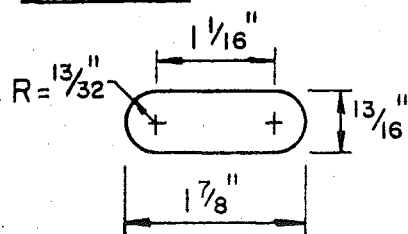


Fig. 3 Test C7



Slot Detail



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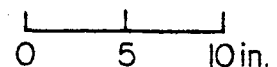
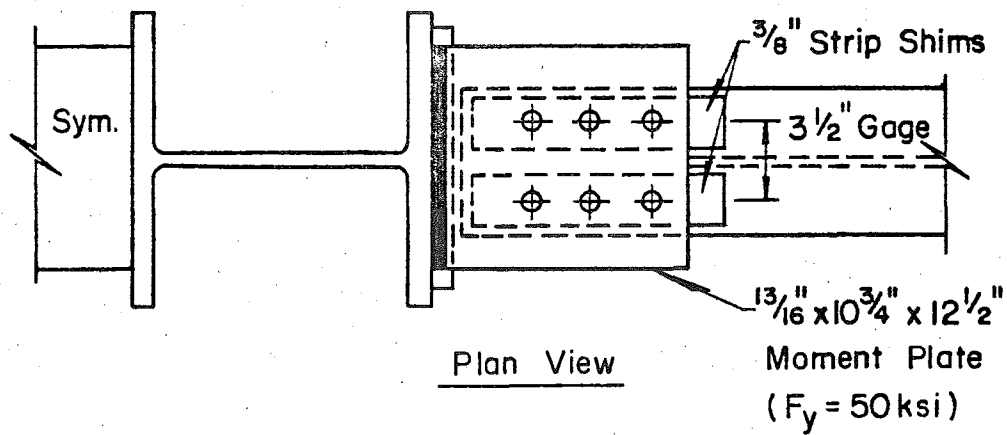
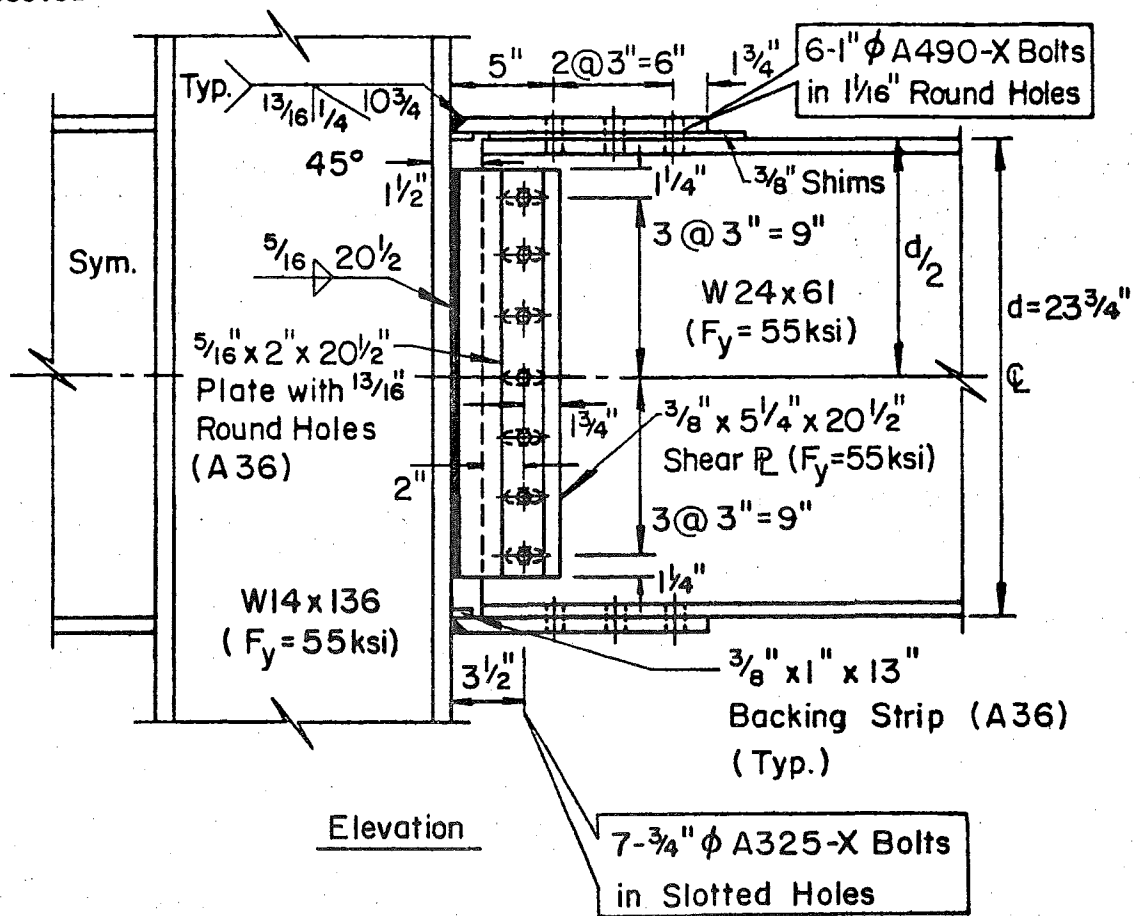
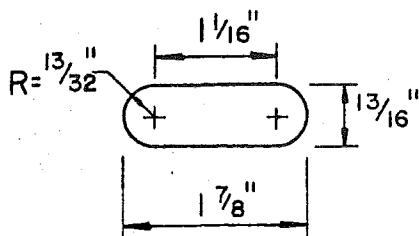


Fig. 4 Test C8





### Slot Detail



Scale:

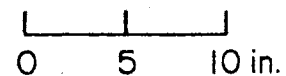


Fig. 5 Test C9

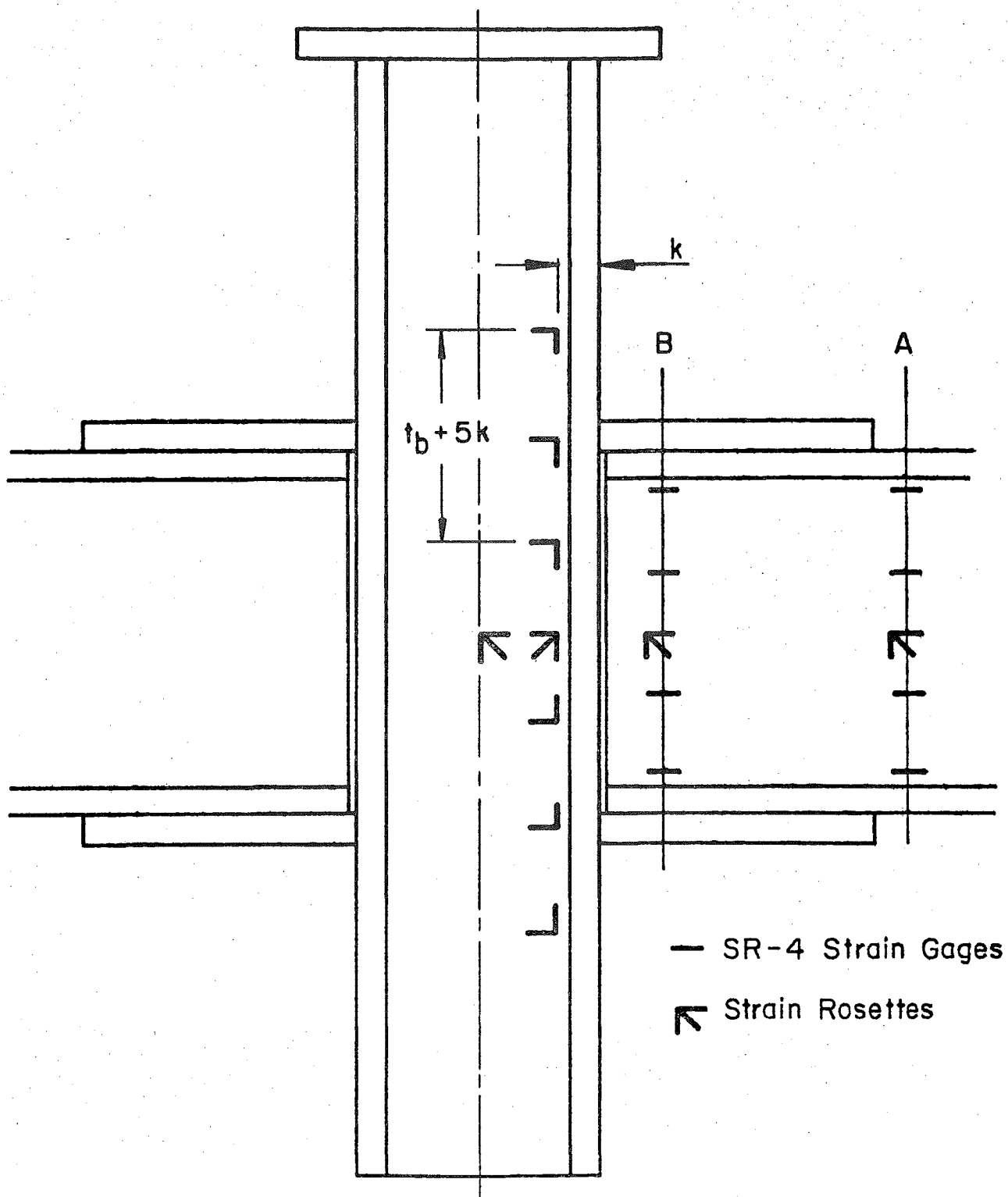


Fig. 6. Strain Gage Locations

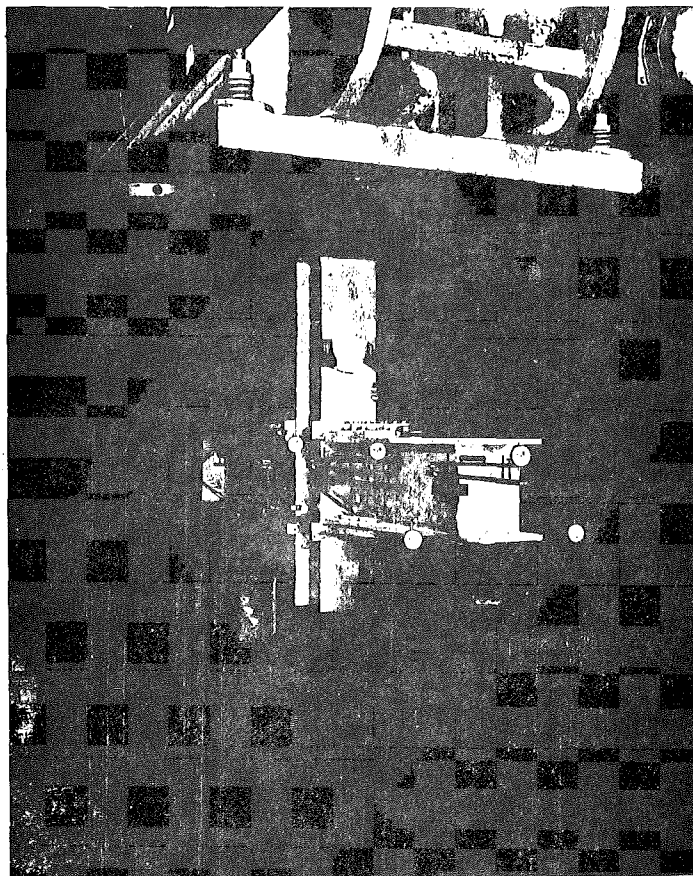


Fig. 7 Test Setup

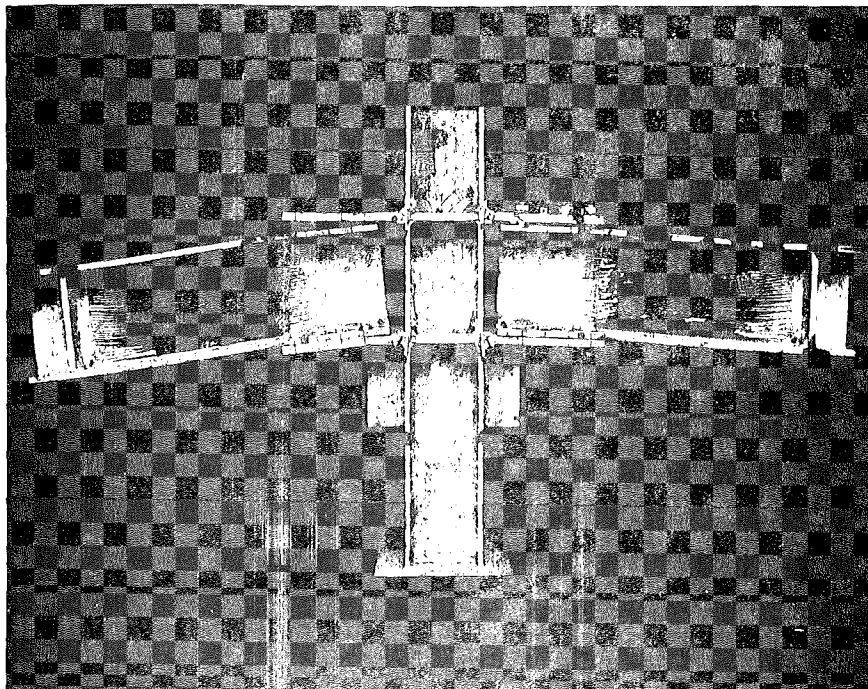


Fig. 8 Overall View of C6

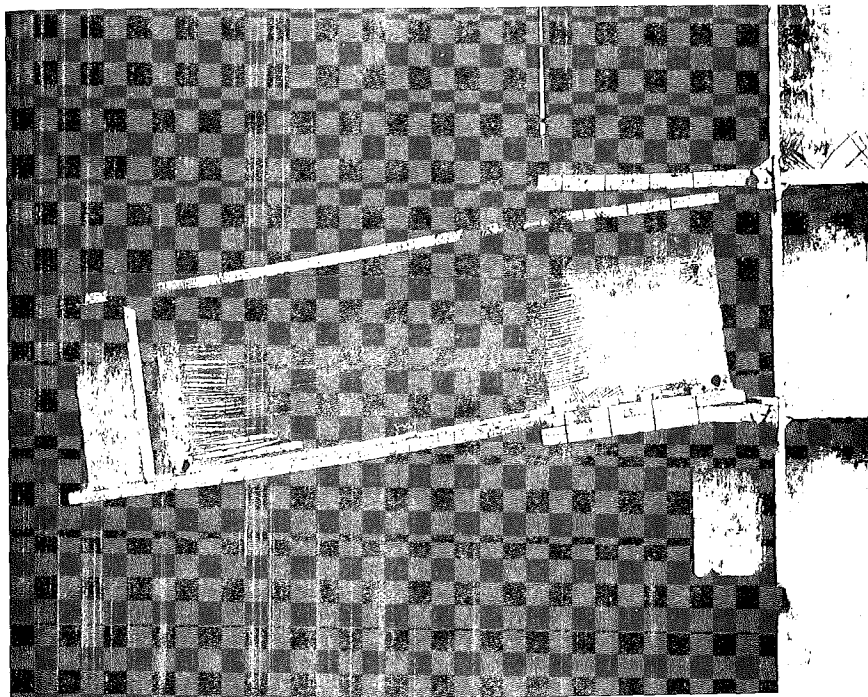


Fig. 9 View of C6 Beam

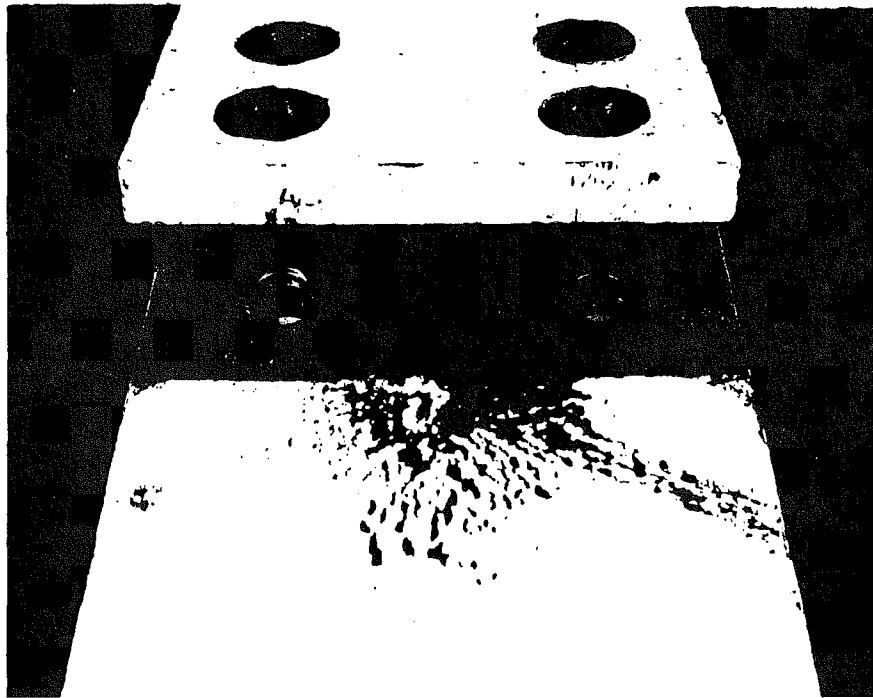


Fig. 10 C6 Beam Flange

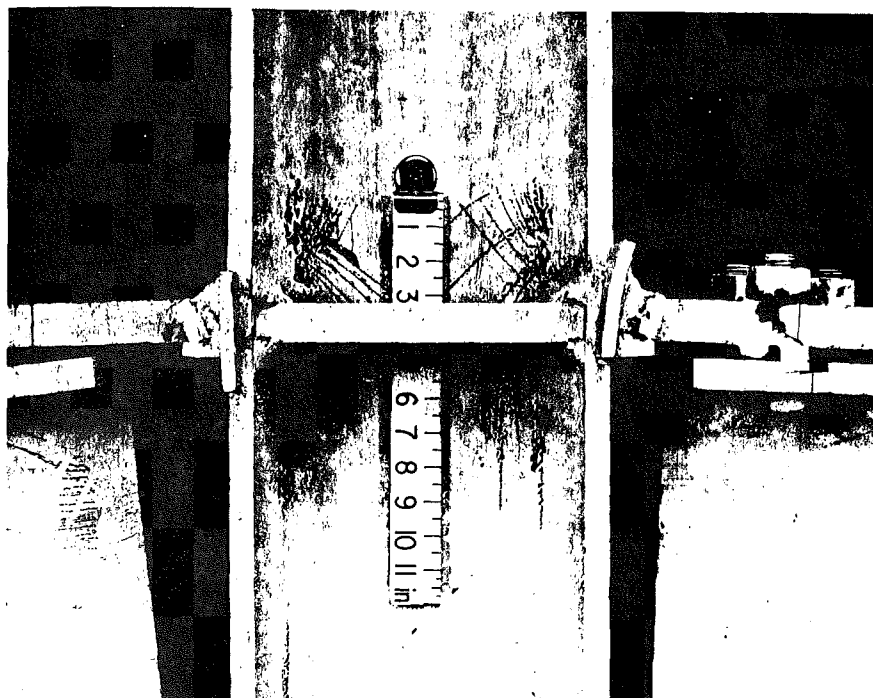


Fig. 11 Tension Panel Zone of C6

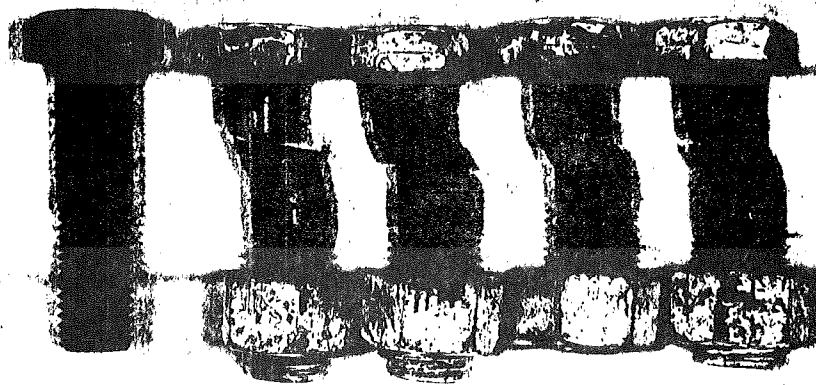


Fig. 12 Sheared Bolts of Connection C6

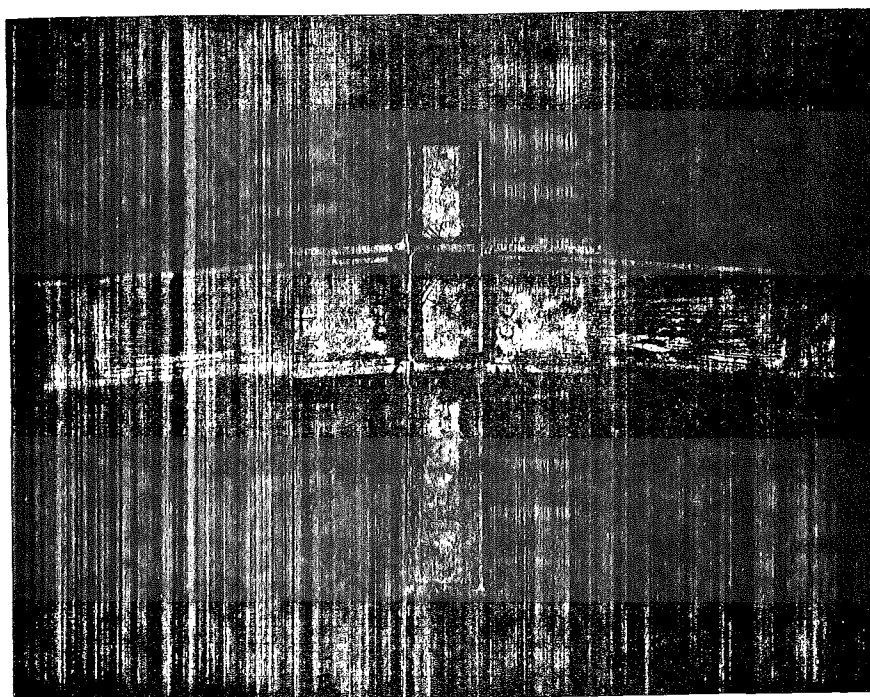


Fig. 13 Overall View of C7

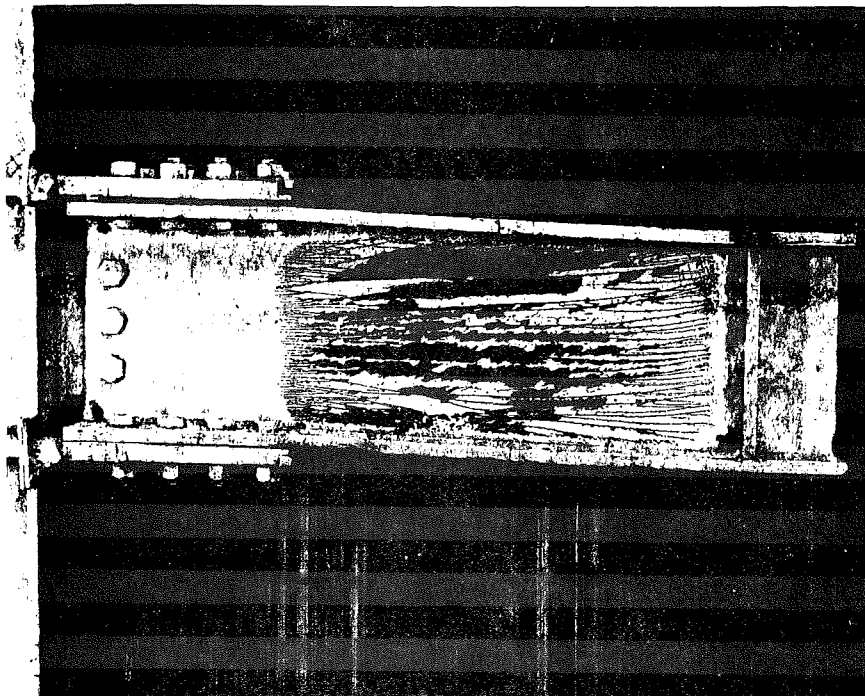


Fig. 14 View of C7 Beam

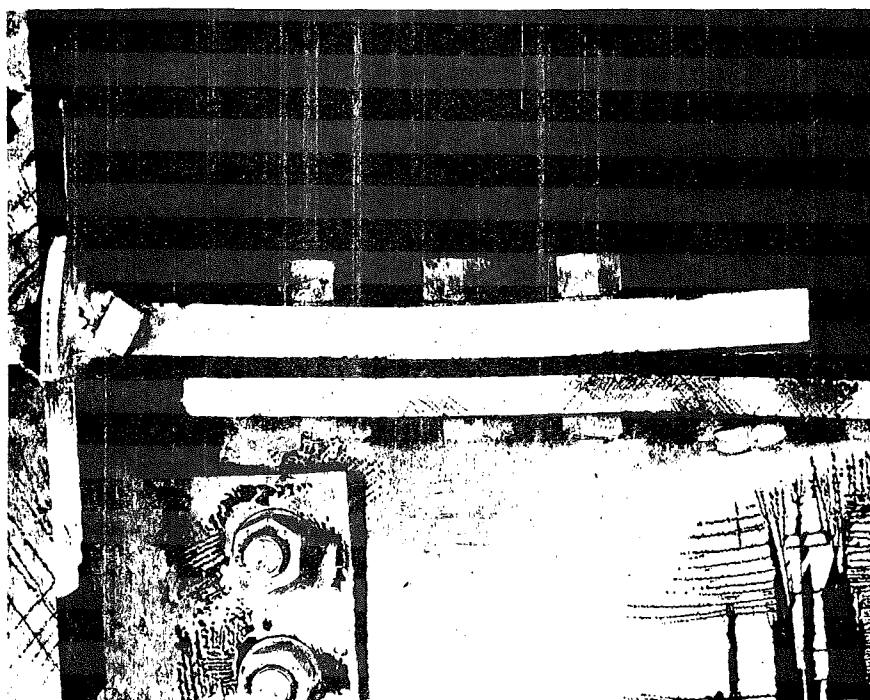


Fig. 15 Prying Action of C7 Moment Plate

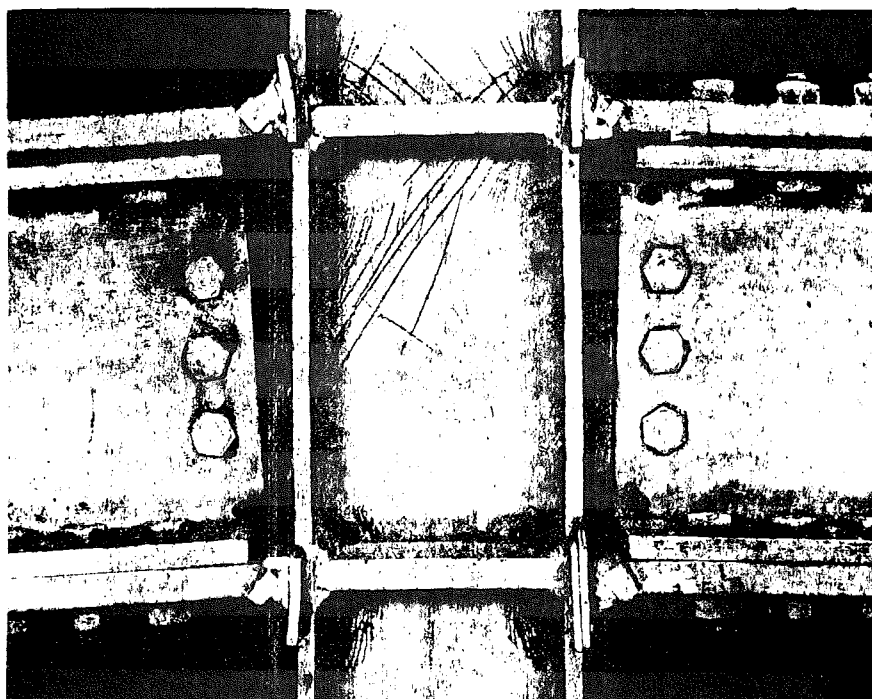


Fig. 16 Panel Zone of Connection C7



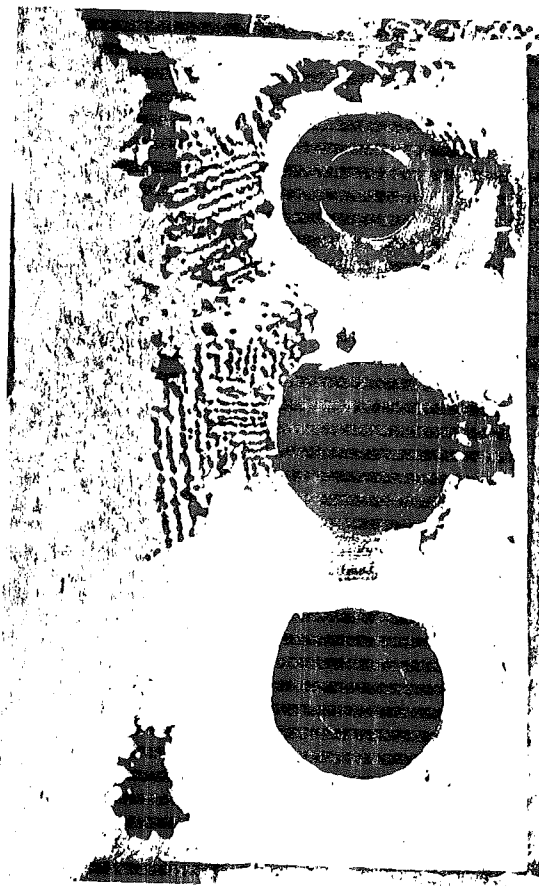


Fig. 17 Beam Web Shear Plate of C7

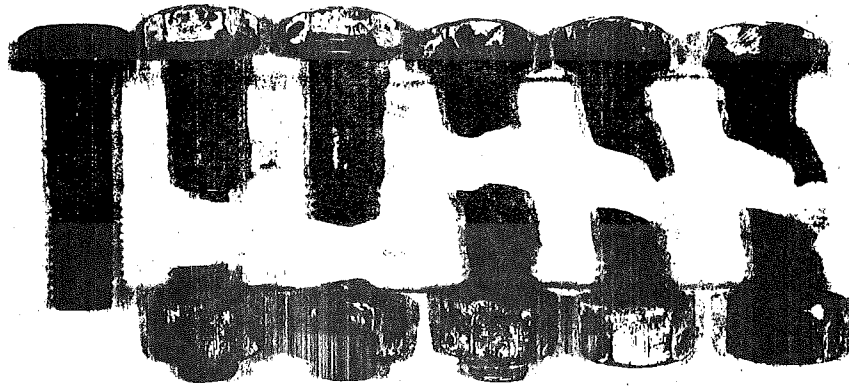


Fig. 18 Bolts at Failure of C7

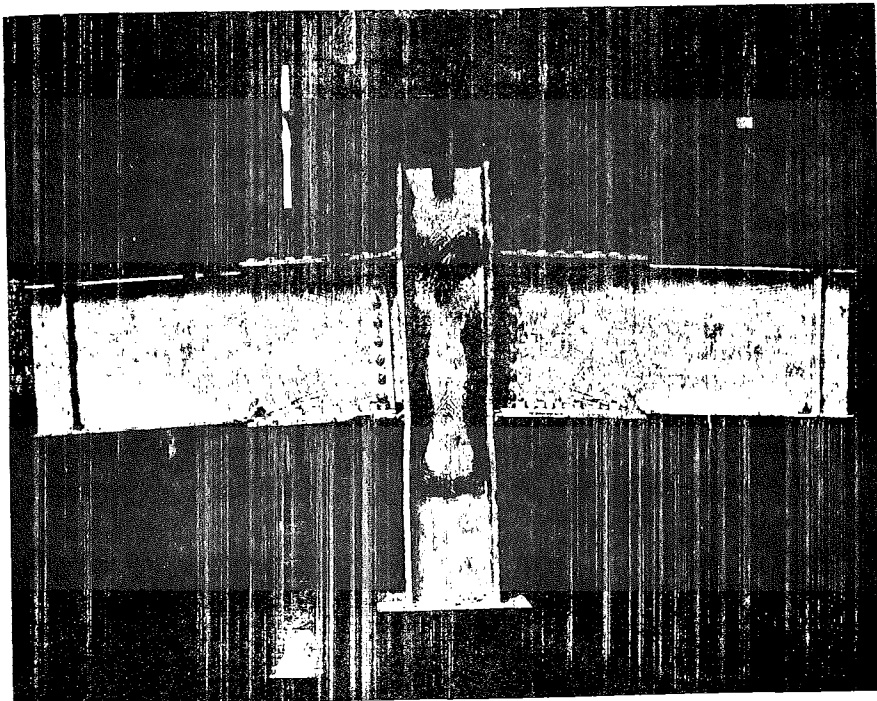


Fig. 19 Overall View of C8

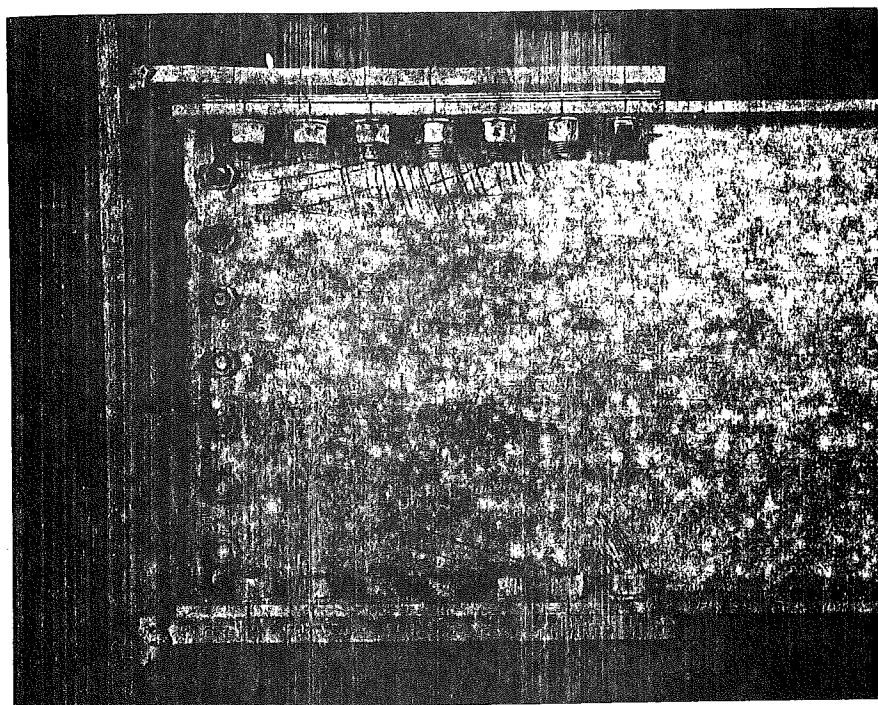


Fig. 20 View of C8 Beam



Fig. 21 Panel Zone of C8

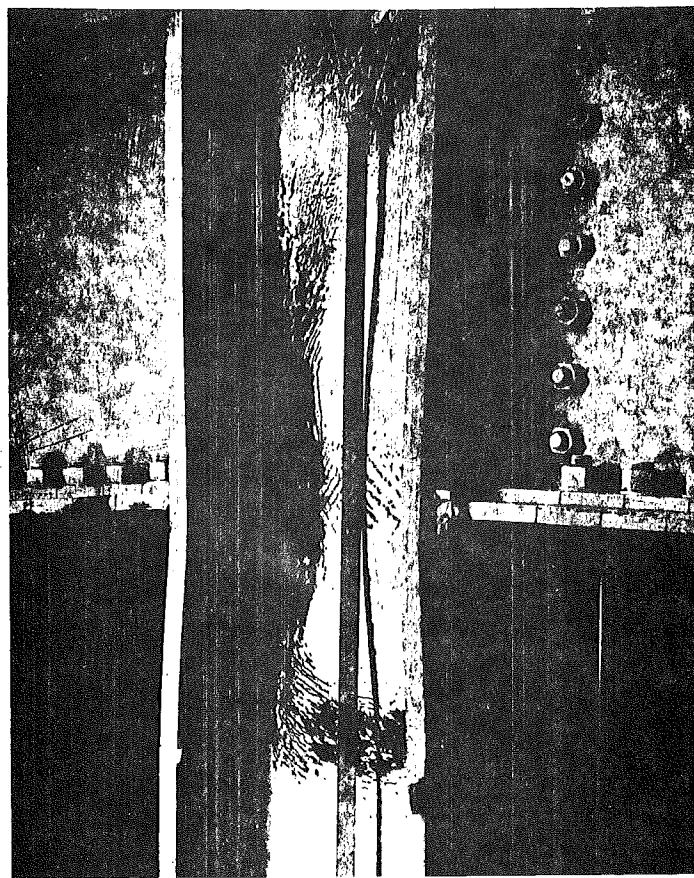


Fig. 22 View Showing Buckling of C8 Column Web



Fig. 23 Area of Bolt Failure on C9

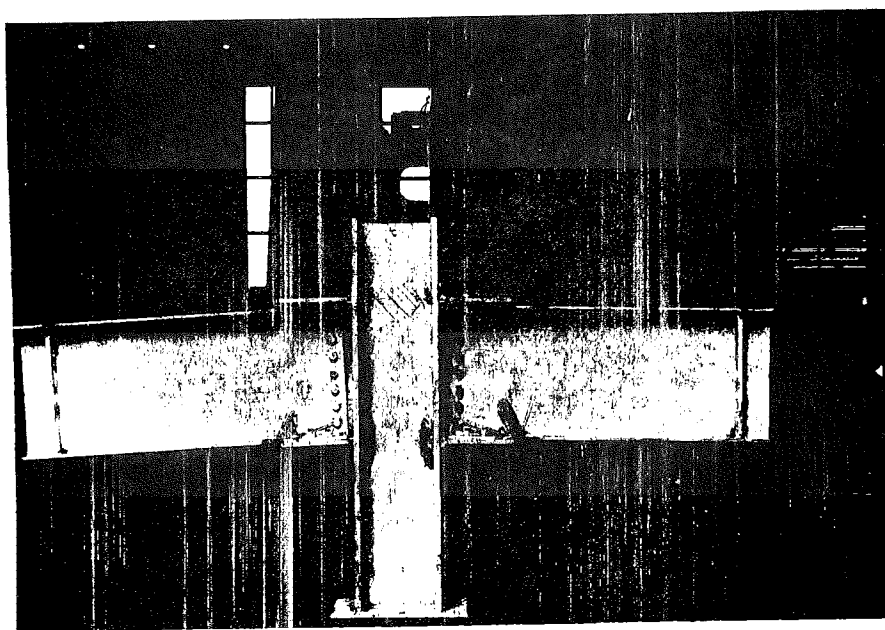


Fig. 24 Overall View of C9



Fig. 25 Panel Zone of Connection C9

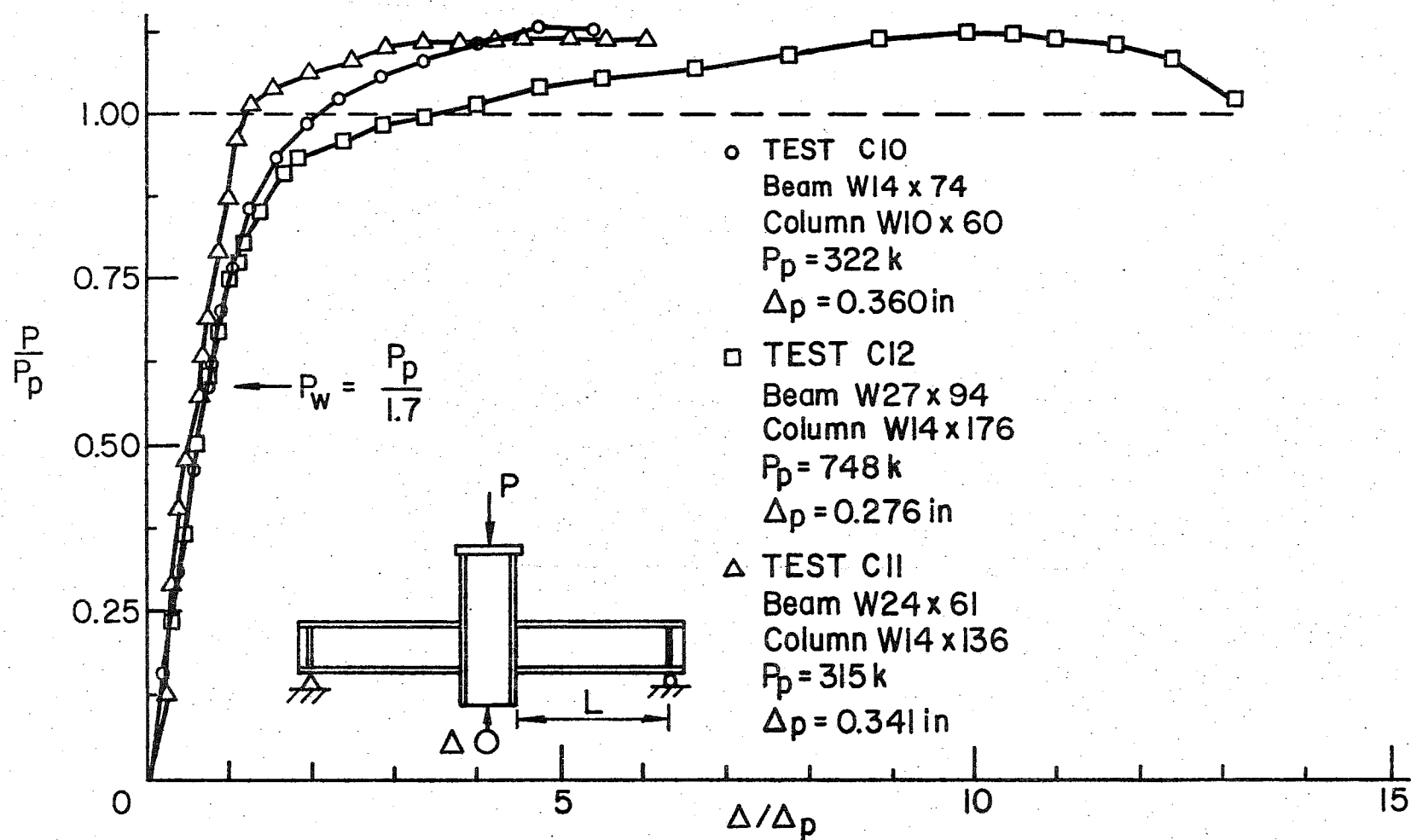


Fig. 26 Load-Deflection Curves--Fully Welded Connections



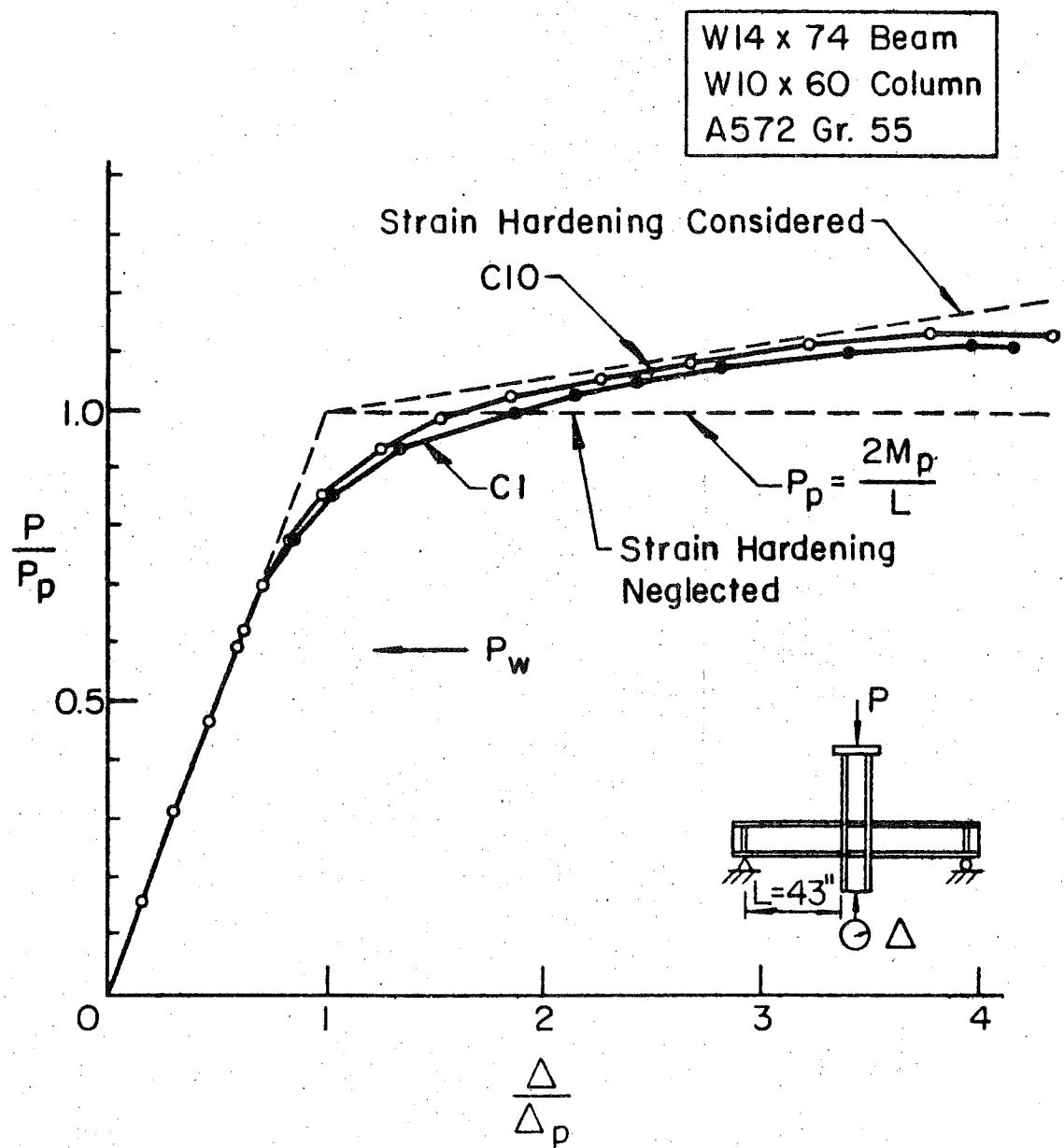


Fig. 27 Load-Deflection Curves--C10, C1 (Ref. 2)

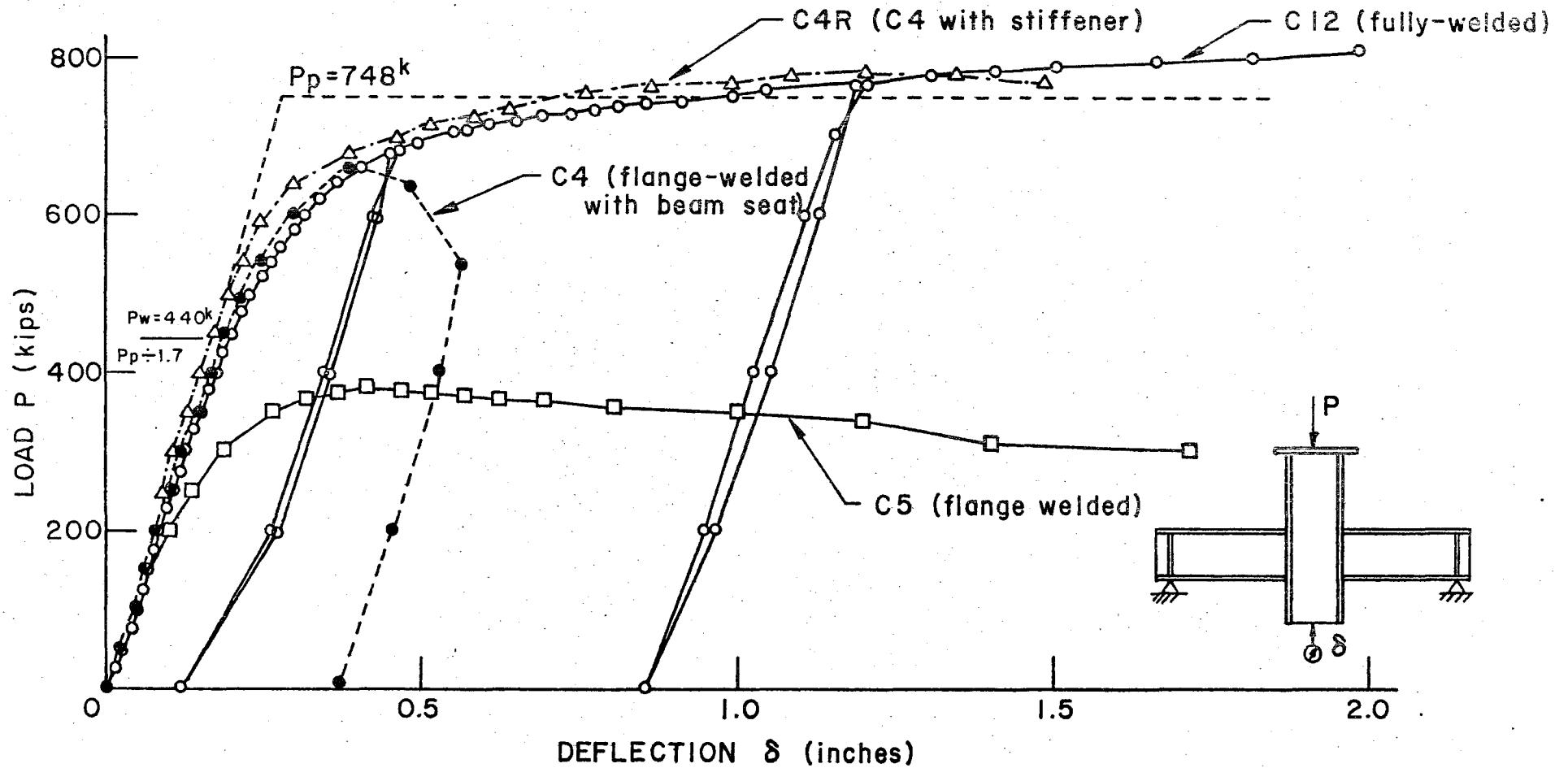


Fig. 28 Load-Deflection Curves--C11, C4, C5 (Ref. 4)

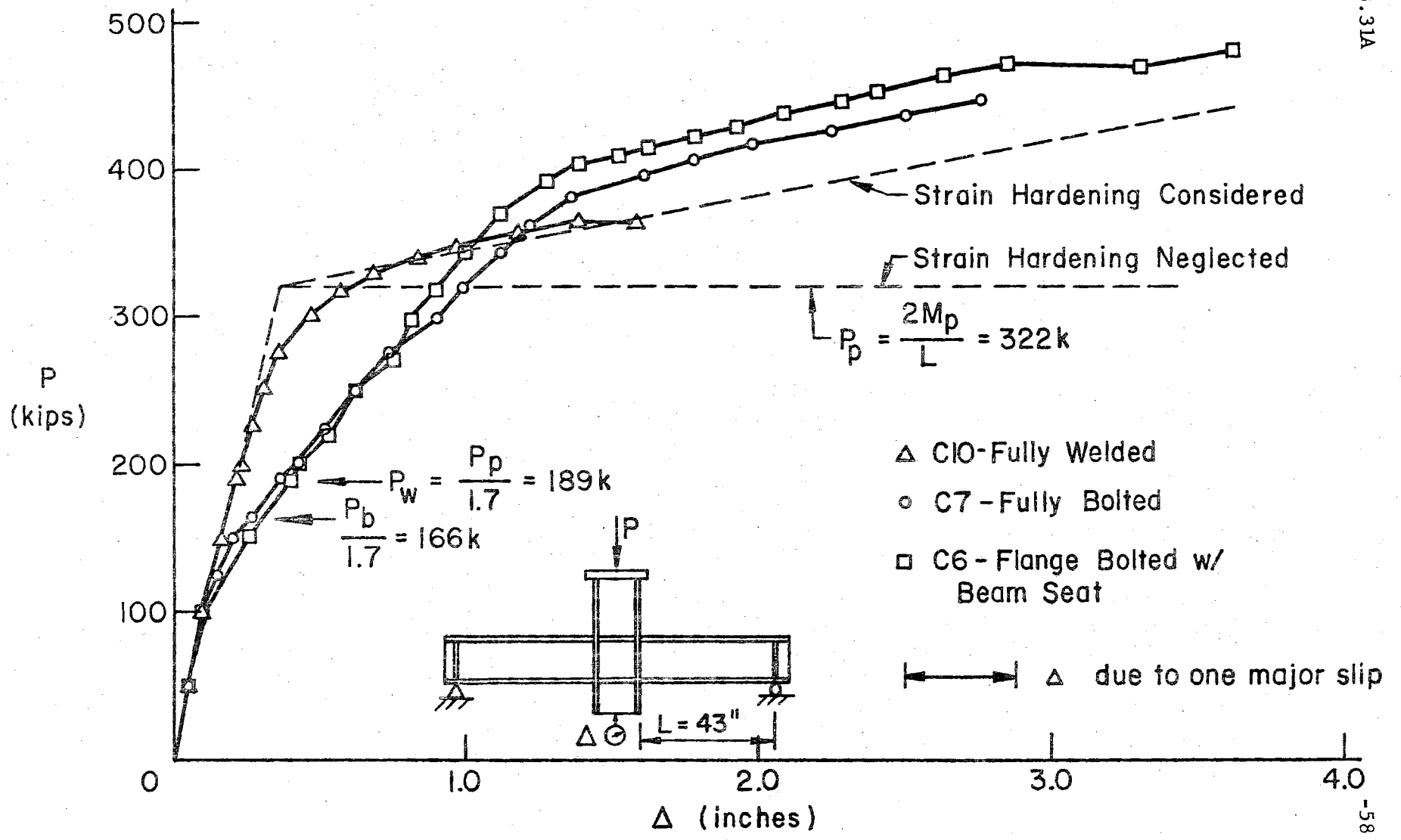


Fig. 29 Load-Deflection Curves--C6, C7

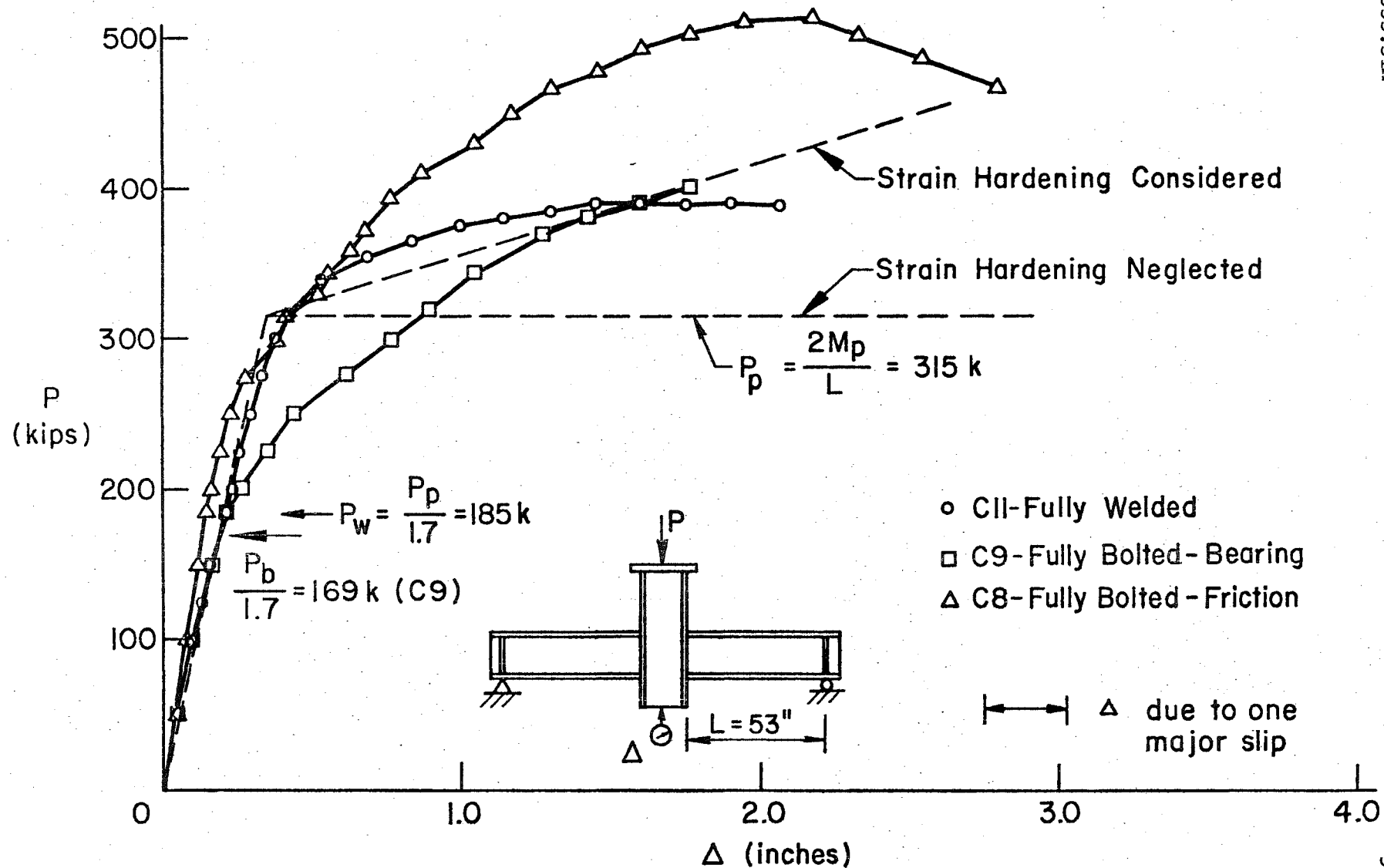
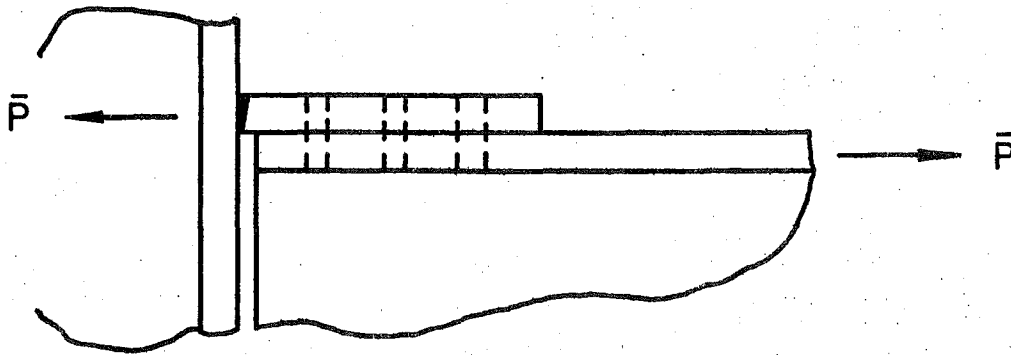


Fig. 30 Load-Deflection Curves--C8, C9



$$\bar{P}_s = mn T_i k_s$$

where  $\bar{P}_s$  = axial load causing slip

$m$  = number of shear planes

$n$  = number of fasteners

$T_i$  = initial clamping force

$k_s$  = slip coefficient

Fig. 31 Lap Joint Idealization

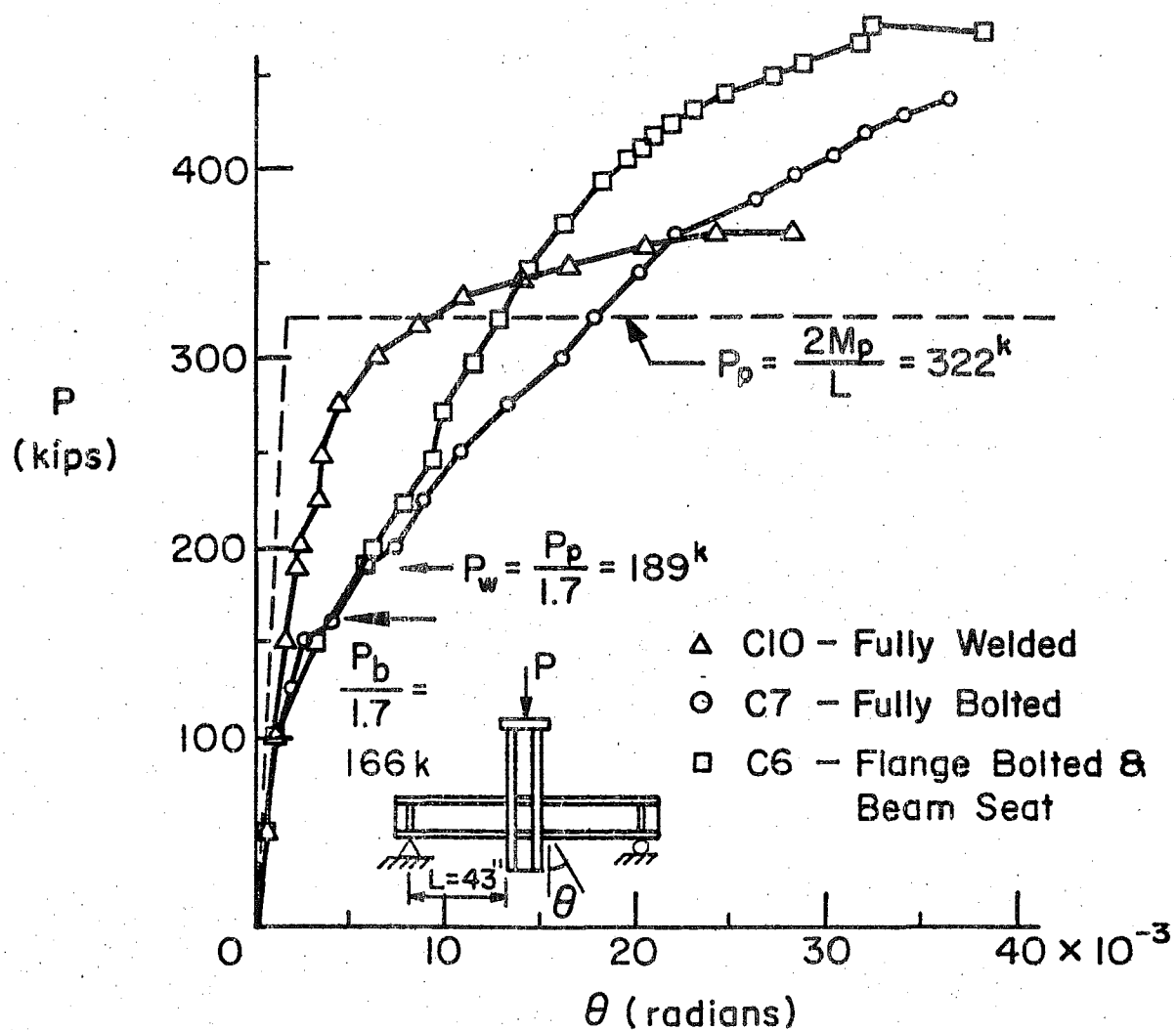


Fig. 32 Load-Rotation Curves--C6, C7

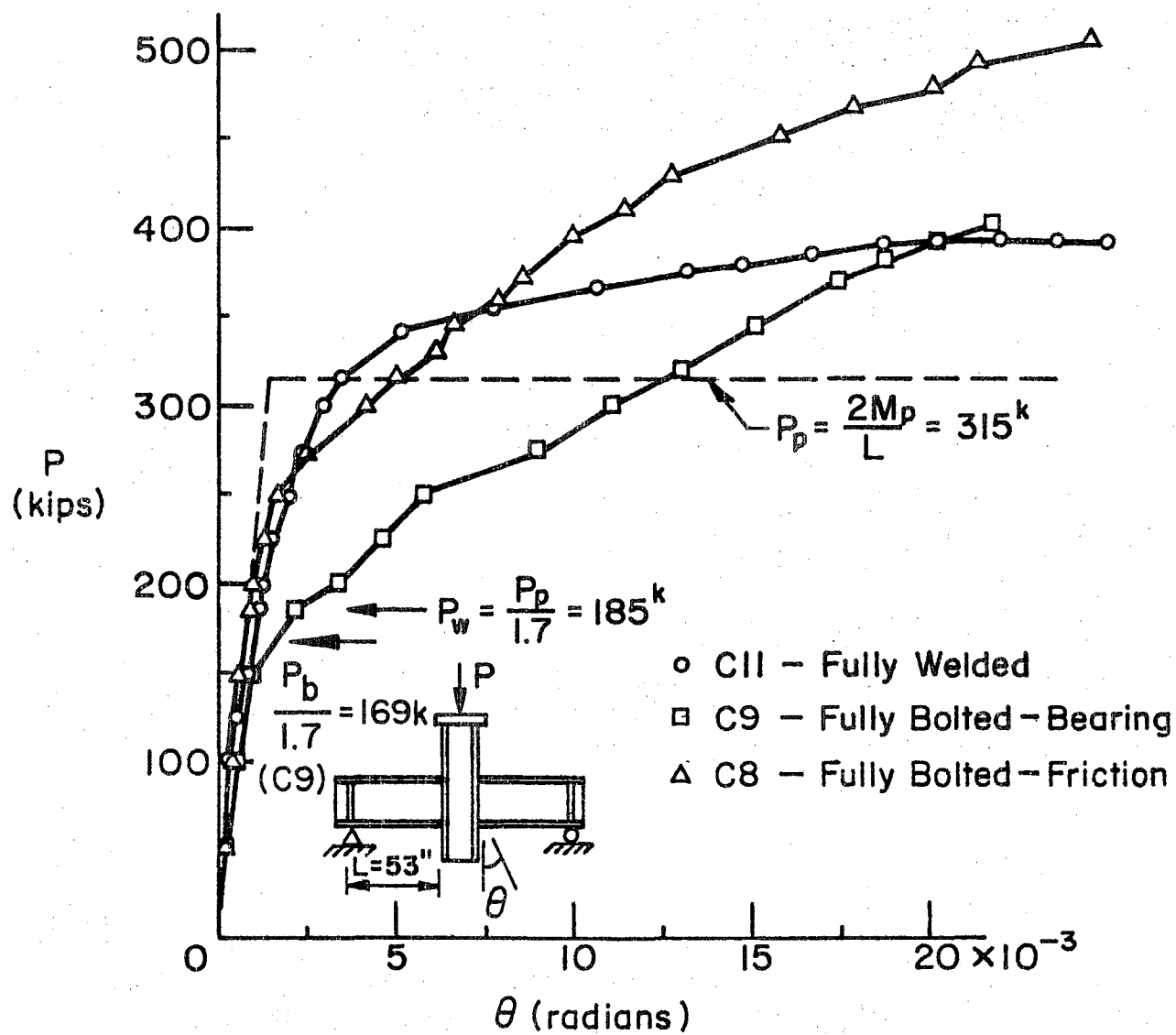


Fig. 33 Load-Rotation Curves--C8, C9

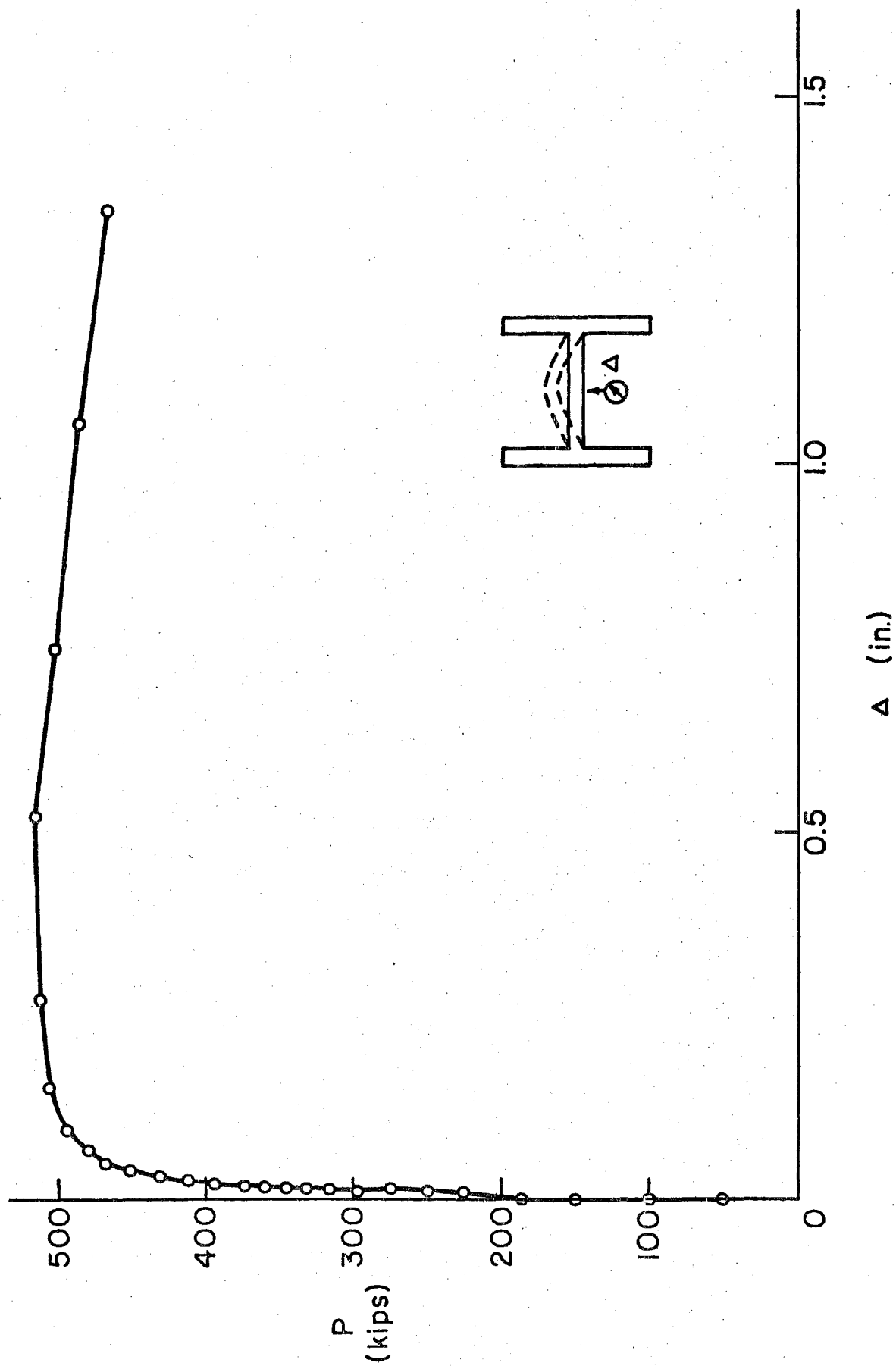


Fig. 34 Web Buckling of C8



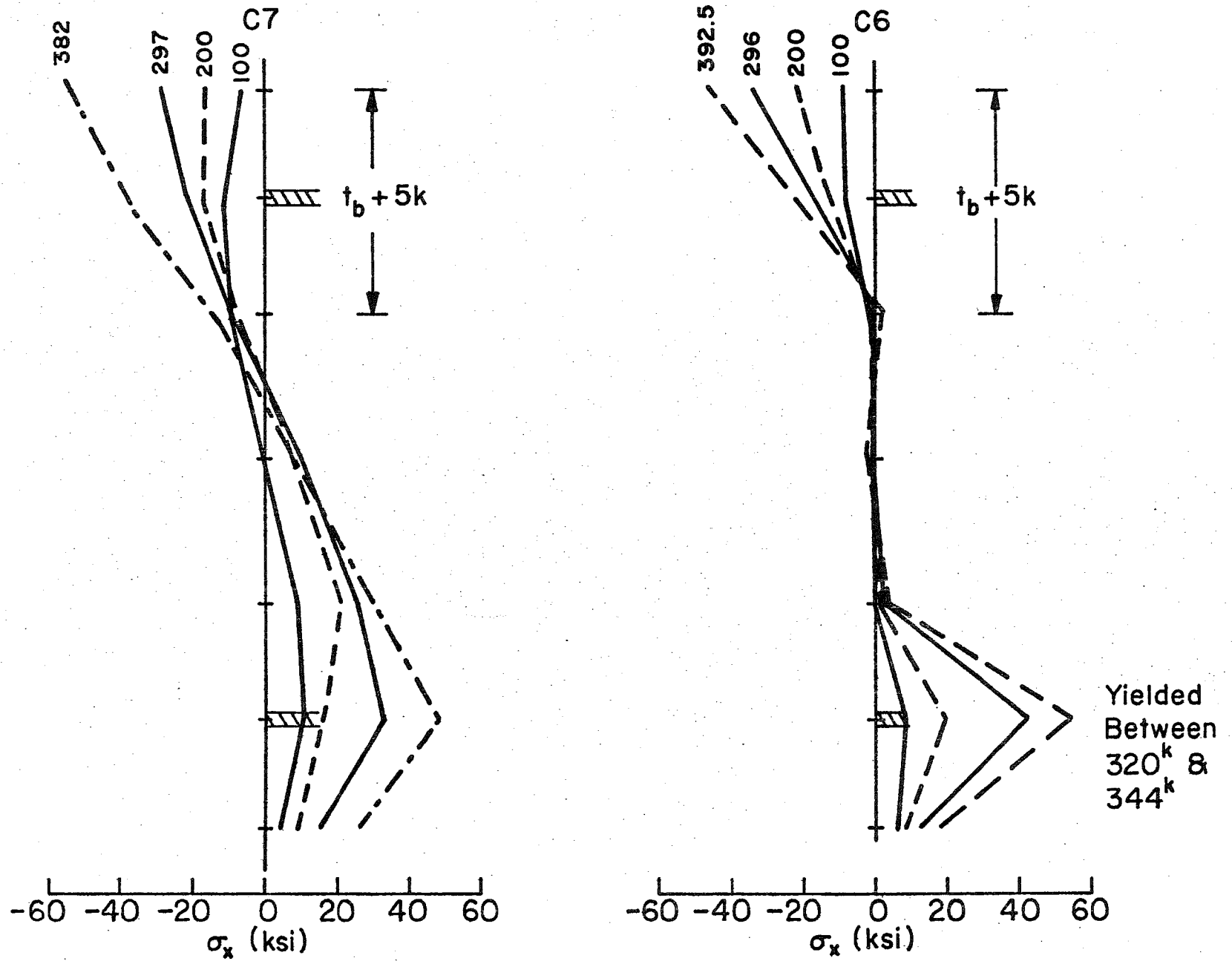


Fig. 35 Variation of Horizontal Stress ( $\sigma_x$ ) Along Column Innerface--C6, C7

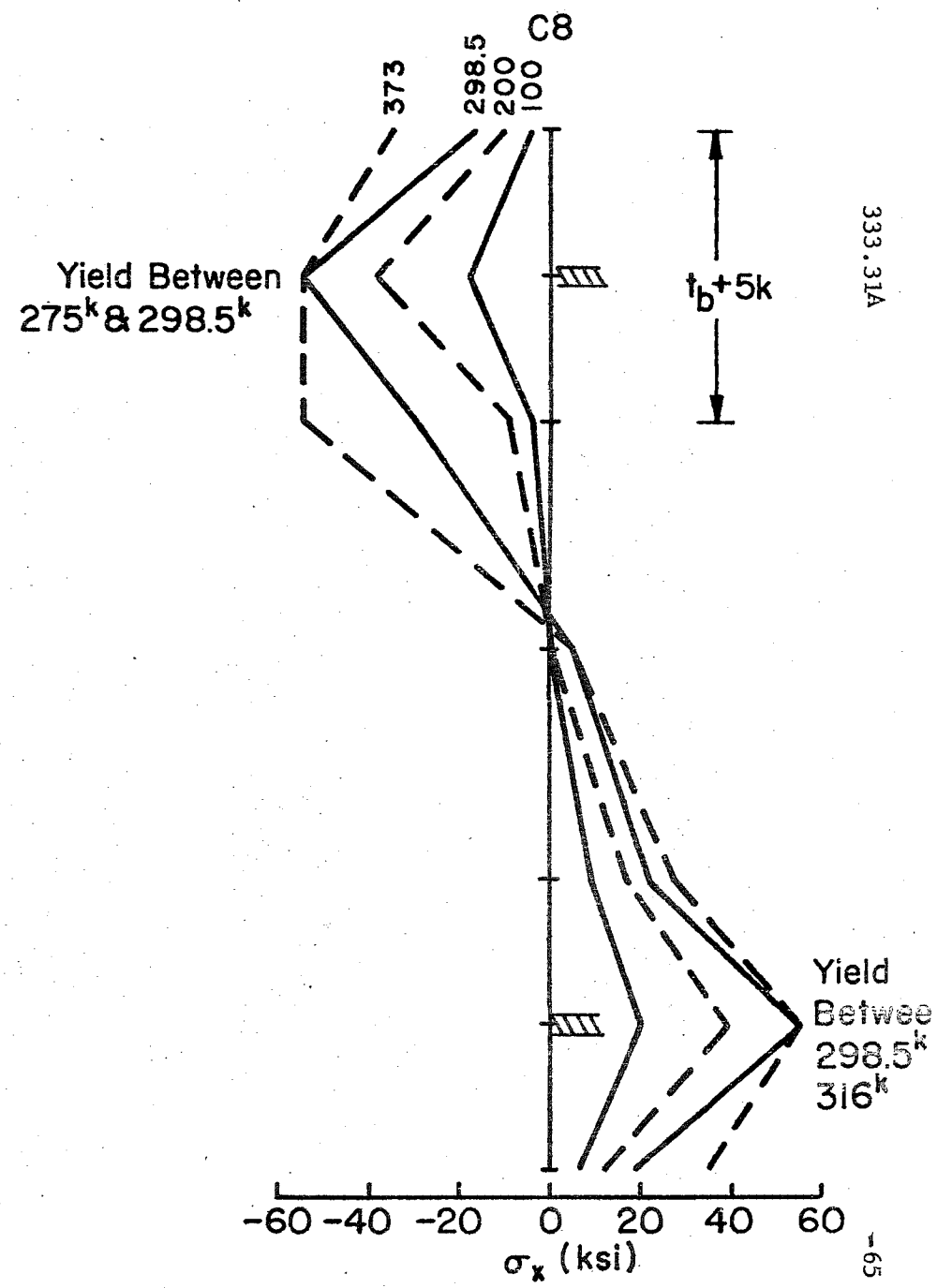
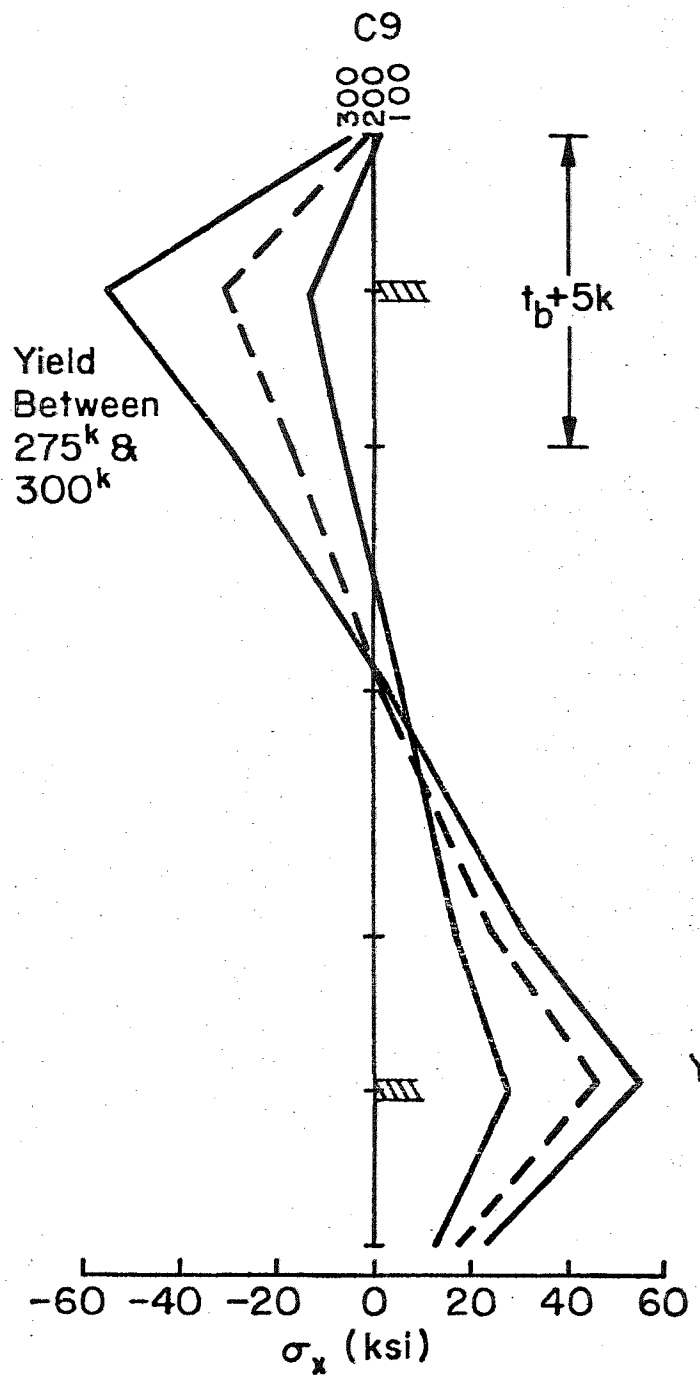


Fig. 36 Variation of Horizontal Stress ( $\sigma_x$ ) Along Column Innerface--C8, C9

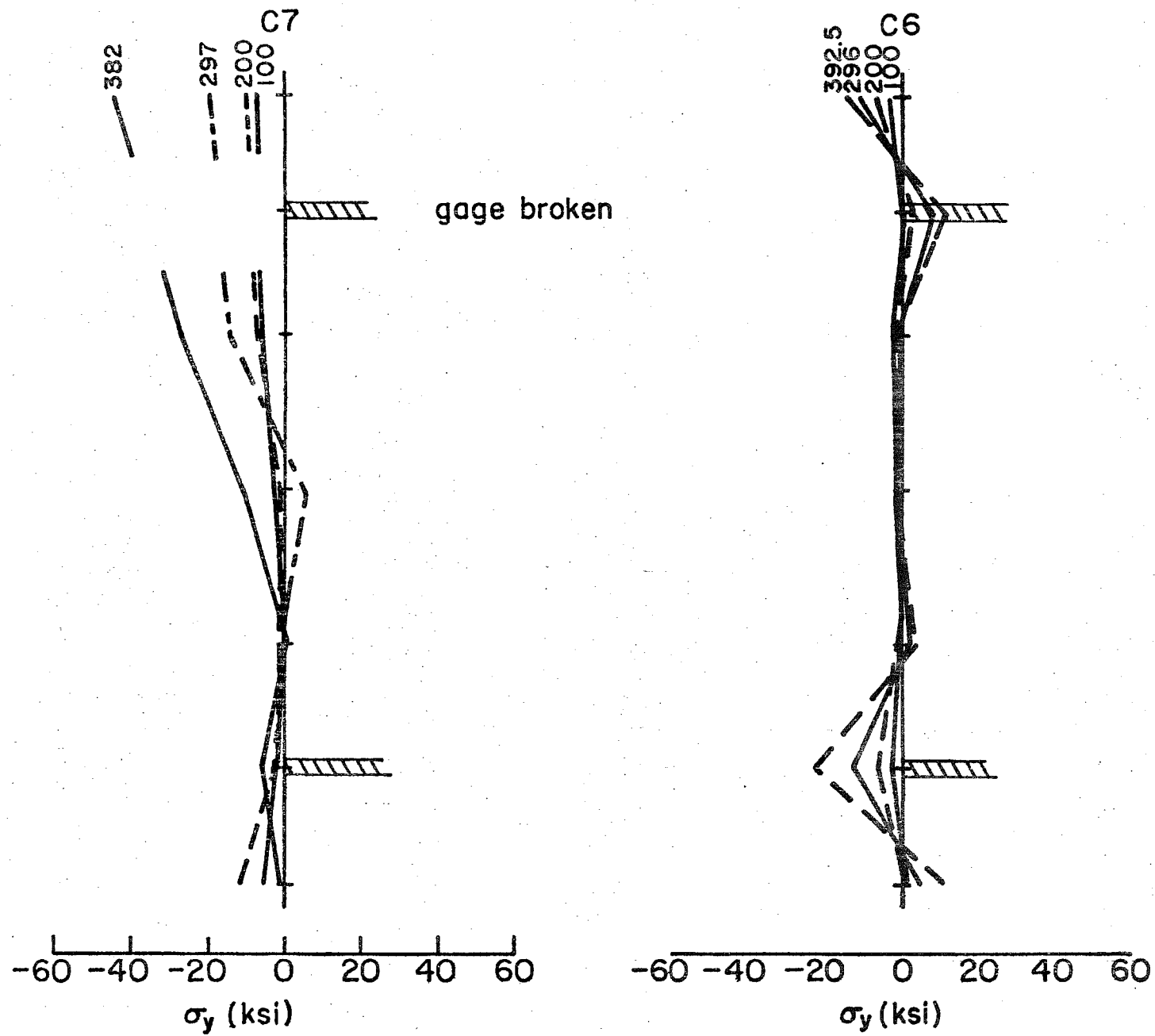


Fig. 37 Variation of Vertical Stress ( $\sigma_y$ ) Along Column Innerface--C6 C7

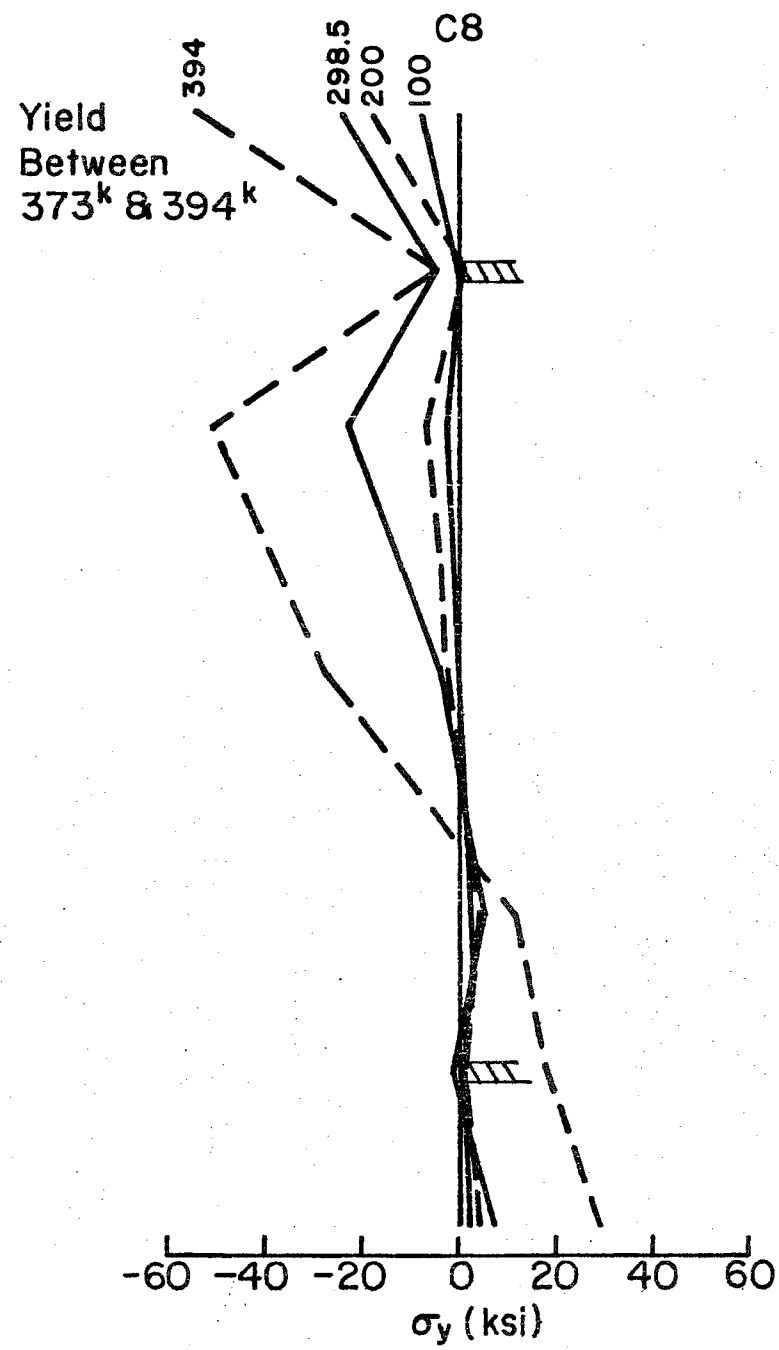
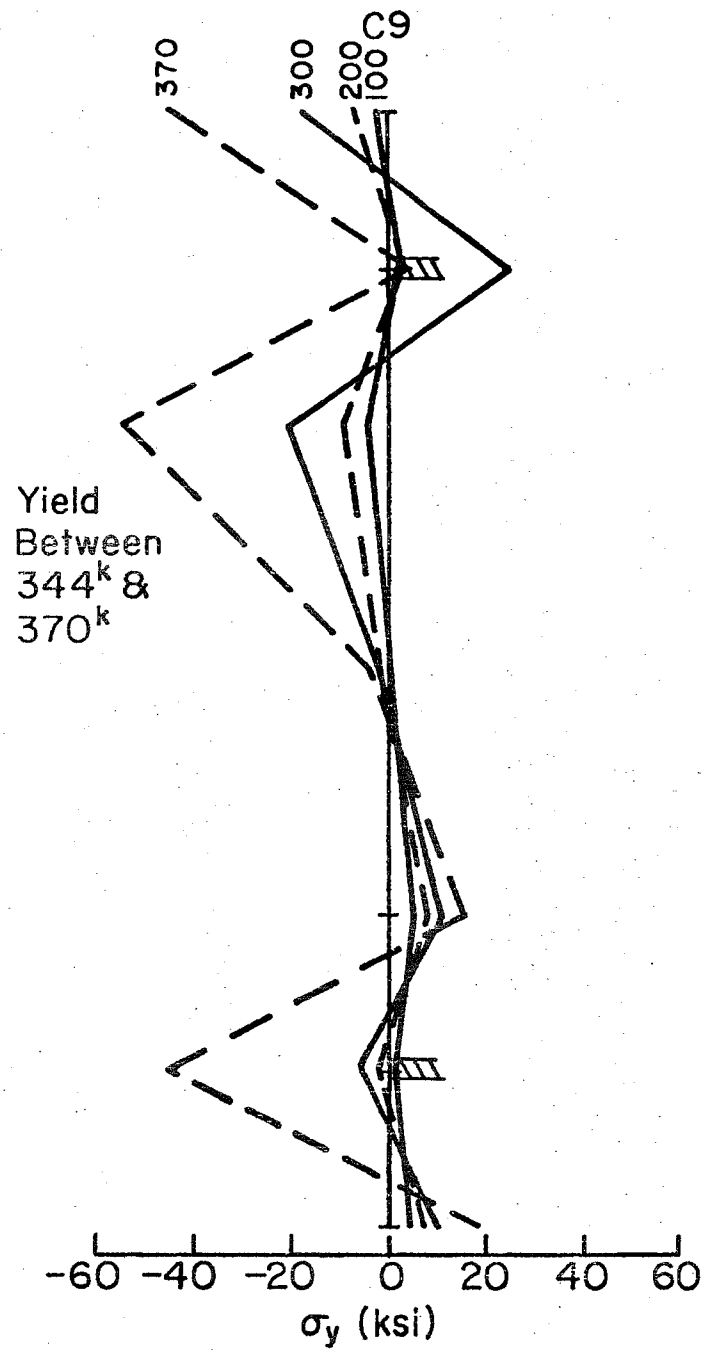


Fig. 38 Variation of Vertical Stress ( $\sigma_y$ ) Along Column Innerface--C8, C9

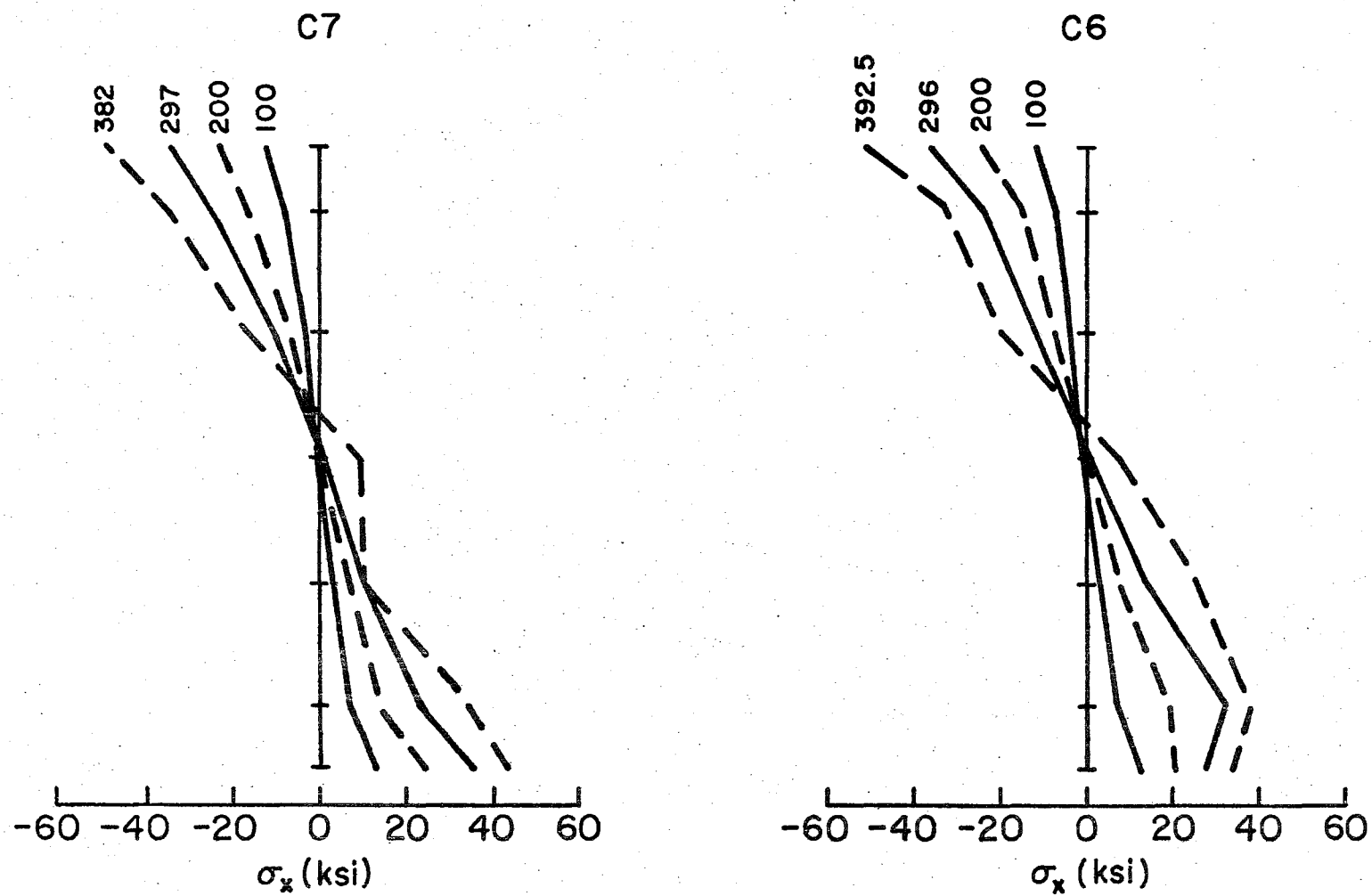


Fig. 39 Variation of Horizontal Stress ( $\sigma_x$ ) in Beam at End of Moment-Plate--C6, C7

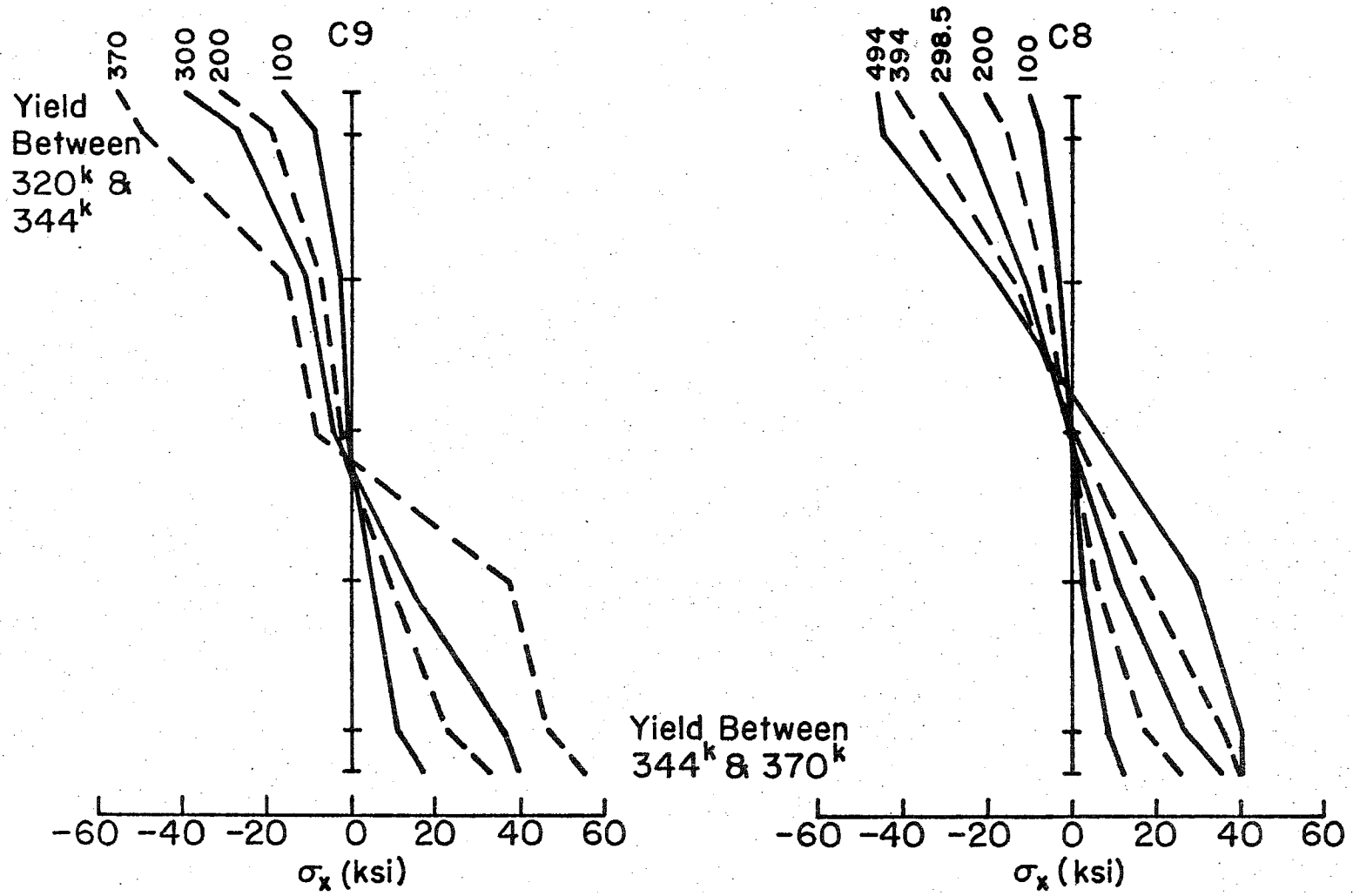


Fig. 40 Variation of Horizontal Stress ( $\sigma_x$ ) in Beam at End of Moment Plate--C8, C9

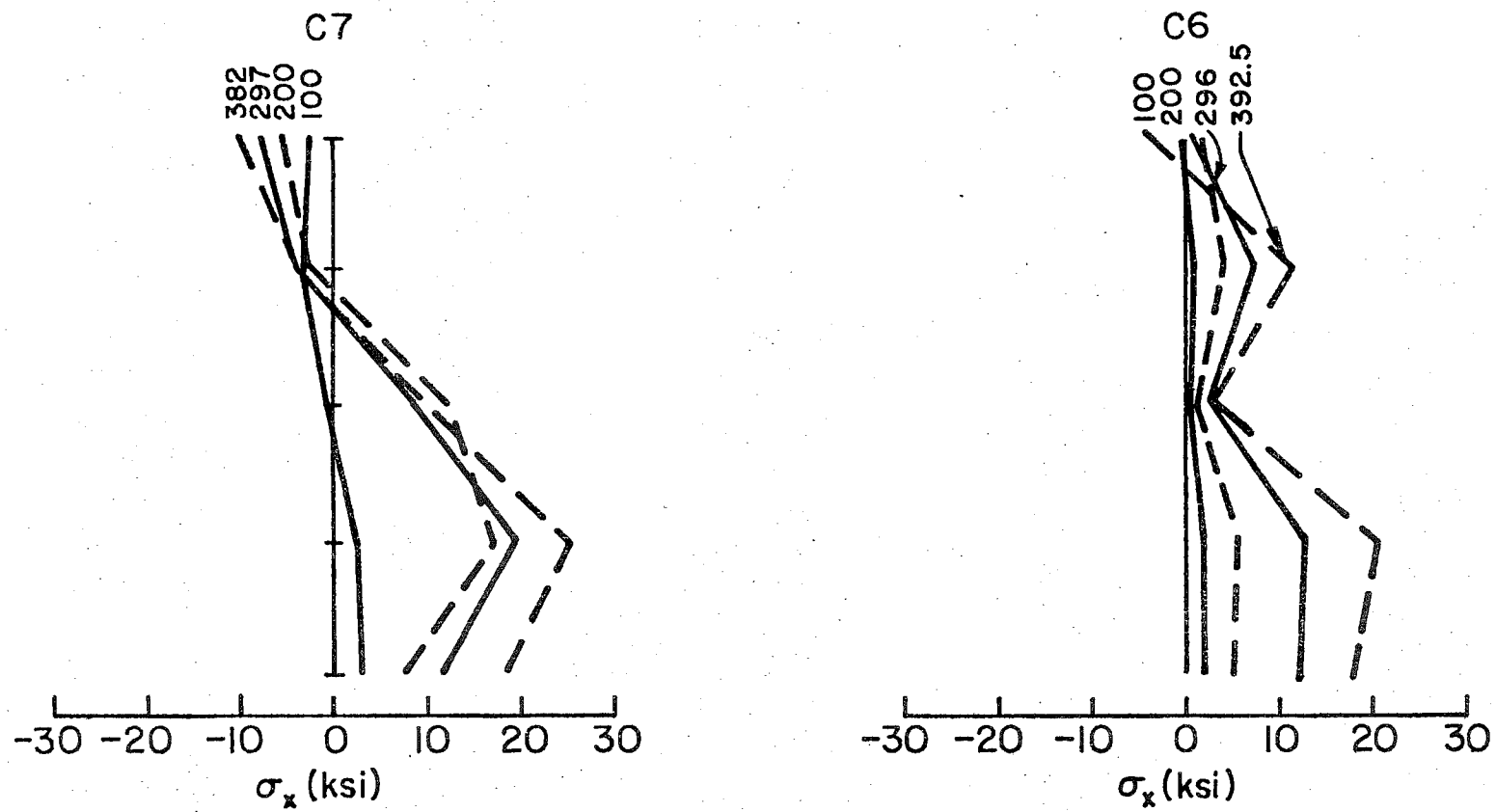


Fig. 41 Variation of Horizontal Stress ( $\sigma_x$ ) in Beam at Column Face--C6, C7

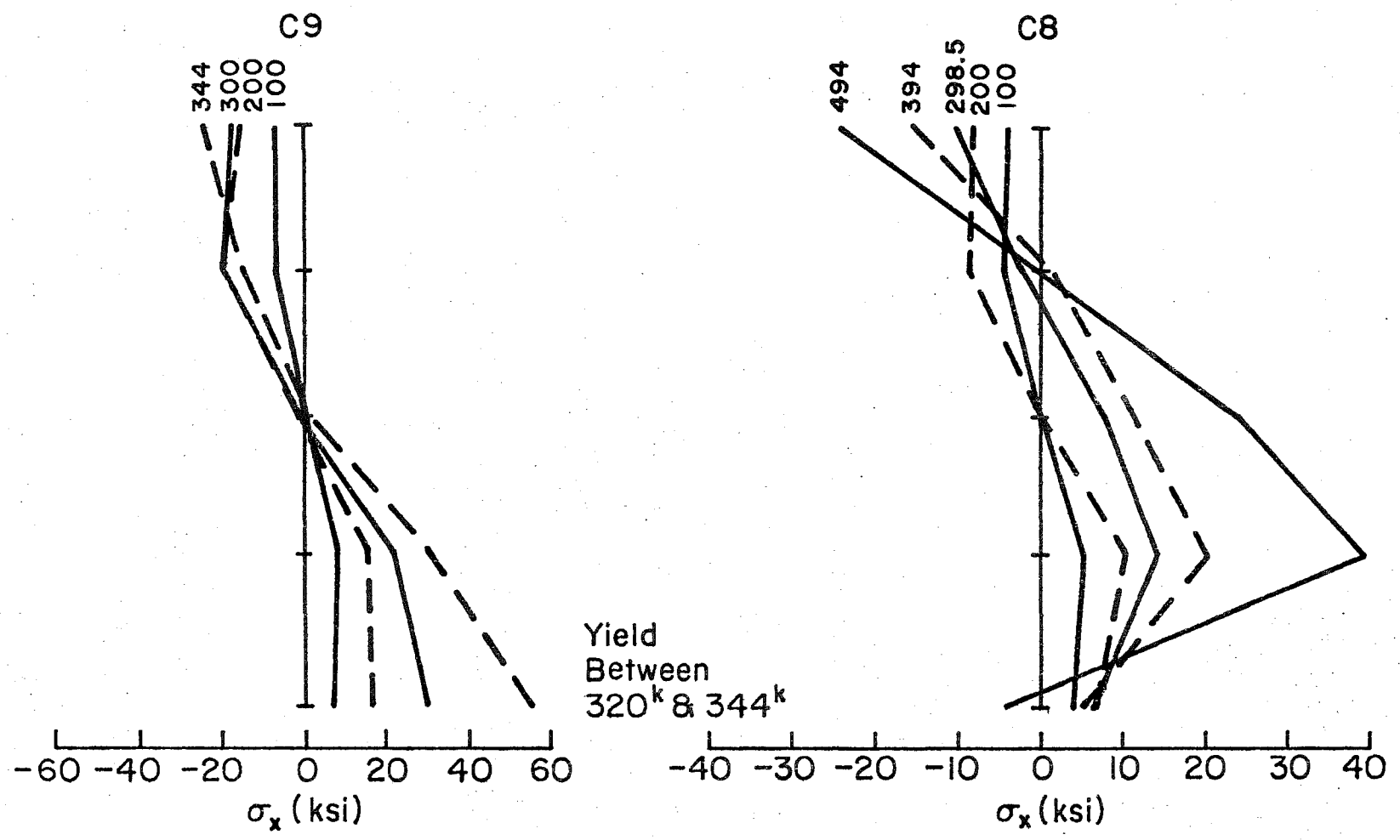


Fig. 42 Variation of Horizontal Stress ( $\sigma_x$ ) in Beam at Column Face--C8, C9



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